ANALYSIS OF STATICALLY INDETERMINATE BUILDING TRUSSES BY THE APPLICATION OF MAXWELL'S THEOREM OF RECIPROCAL DISPLACEMENTS; AND THE DEVELOPMENT OF EMPIRICAL FORMULAS FOR THE REACTIONS AND MOMENTS

BY

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THESIS

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MAY 31, 1920.

I HEREBY RECOMMEND THAT THE THESIS PREPARED UNDER MY SUPERVISION BY SAMUEL RUSSELL OFFUTT.

ENTITLED ANALYSIS OF STATICALLY INDETERMINATE BUILDING TRUSSES BY THE APPLICATION OF MAXWELLS THEOREM OF RECIPROCAL DISPLACEMENTS; AND THE DEVELOPMENT OF EMPIRICAL FORMULAS FOR THE REACTIONS AND RESISTING MOMENTS

BE ACCEPTED AS FULFILLING THIS PART OF THE REQUIREMENTS FOR THE DEGREE OF MASTER OF SCIENCE IN CIVIL ENGINEERING.

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Recommendation concurred in*

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______________________	on
______________________	Final Examination*

*Required for doctor's degree but not for master's

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I.

INTRODUCTION.

A. Preliminary.

A building truss when subject to horizontal loading is a statically indeterminate structure. Two classes of this type of structure are herein considered.

Class I. Columns assumed to resist no bending moment at their bases.

A solution of this problem necessitates the determination of four unknowns, viz:- the horizontal and vertical reactions at each column base. The three equations of static equilibrium being obviously inadequate, it is customary in designing practice to obtain the necessary fourth equation by the assumption of equal horizontal reactions.

Class II. Columns assumed as fixed at their bases and thereby capable of resisting bending moment.

This problem presents six unknowns, viz:- the horizontal and vertical reactions and the resisting moment at each column base. Here the three equations of static equilibrium must be augmented by three other equations. In this case, also, practice has circumvented the difficulty by arbitrary assumptions, viz:

1. Assuming equal horizontal reactions.
2 & 3. Assuming the location of the point of contraflexure or zero bending moment in each column.

All authorities seem to agree on the assumptions of equal horizontal reactions, but they differ regarding the assumptions for the location of the point of contraflexure. The assumptions with regard to the location of this point are of two classes,

1. Arbitrary Assumptions. This is the method used generally in designing practice. The point of contraflexure is usually assumed midway between the column base and the foot of the knee brace; but where it is quite evident that the columns are not rigidly fixed at their bases, the point of contraflexure is often lowered to one-third or one-fourth the distance between column base and knee brace.

2. By Formulas Based on Other Assumptions. This method may be subdivided into three classes according to the assumptions made.

a. Columns are assumed fixed at their bases, and the horizontal deflections of the top of the column and the foot of knee brace are assumed equal. Under this assumption the point of contraflexure is found by the formula: $y = \frac{d}{2} \times \frac{d + 2h}{2d + h}$;
where, y is the height of the point of contraflexure above the column base, d is the height to the foot of the knee brace and h is the height of the column.

b. Columns are assumed to be fixed at their bases and to be rigidly fixed for the portion above the foot of knee brace. In accordance with this assumption the point of contraflexure

is midway between the column base and foot of knee brace. This assumption therefore is in accord with the arbitrary assumption.

These methods are given by Ketchum in "Steel Mill Buildings," Third Edition, p. 87.

c. Columns are assumed to be rigidly fixed at their bases and tops. In accordance with this assumption the point of contraflexure is midway between top and bottom of column. This method is rarely used as it seems to place the point of contraflexure too high.

This method is given by Burr and Falk in "Influence Lines for Bridges and Roofs," Third Edition, p. 53.

All these assumptions are mere guesses, but some of them have been used so long and consistently that they have gained the sanction of good engineering practice. Heretofore no one has known how any one of these assumptions compares with actual facts in the case.

B. Purpose.

The purpose of this thesis is threefold:

1. To present a rational solution of the problem: a solution based upon the known elastic properties of the material and Maxwell's Theorem of Reciprocal Displacements.

2. To determine how these rational results compare with the results obtained by making the common assumptions.

3. To study the effect on the reactions of variations in different characteristics of the structure and develop empirical formulas for determining the reactions and moments.

C. Scope.

The method of solution presented herein is general and may be applied to any structure consisting of a truss of any type supported on two columns and connected in two places to each column.

The empirical formulas developed are not so general in their application as is the method of solution. Time has not permitted a treatise of the subject, which would include a study of the effect on the reactions of a variation in each and all the characteristics of the structure. However, a type of structure very commonly used in practice has been chosen for this thesis; and the results show the effect of variations in those characteristics of the structure, which appreciably affect the reactions and moments.

The writer does not presume to draw general conclusions from meager data. Rather the intention is to make an intensive study of certain phases of the problem and find out where more time might be most profitably spent.

D. Acknowledgment.

This thesis was written under the direct supervision of C. A. Ellis, Professor of Structural Engineering in the Department of Civil Engineering. Some of the investigation herein reported was made by the writer as part of his work as graduate Assistant in the Engineering Experiment Station.

The method presented for the determination of the reactions was formulated by Professor Ellis, who had started work on the problem and almost completed the analysis of one structure under concentrated loads when the writer took up the work. The writer wishes also to gratefully acknowledge his indebtedness to Professor Ellis for his helpful suggestions and hearty co-operation in all the work of the thesis.

II.

METHOD OF SOLUTION.

A. In general.

Maxwell's Theorem of Reciprocal Displacements establishes a mutual relation between any two points of a structure. This theorem, which easily admits of proof, may be stated thus:

If a force P at any point of a structure, as at A, acting in any direction, as horizontal, causes a deflection d of any other point of the structure, as B, in any direction, as vertical; then will the force P acting vertically at B cause a horizontal deflection d at A.

This principal le furnishes an excellent means for determining the reactions of an indeterminate structure, as a mill building truss when subject to horizontal loads. For example: suppose we have given a truss as in Fig. 1 loaded with a horizontal force of one pound at J, and we wish to know the vertical reaction at B caused by this load. The method for determining this reaction is as follows:

Apply a force of one pound vertically at B. (Fig. 13). This force causes the structure to deflect. Find d_B, the vertical displacement of B, and d_J, the horizontal displacement of J. By Maxwell's Theorem, the displacement d_J equals the vertical displacement which B would undergo due to a horizontal load of one pound at J. Now, if one pound at J causes the point

[illegible]

B to displace d_J and one pound at B causes the point B to displace d_B, then the reaction at B caused by one pound at J is

$$R_B = d_J/d_B \times 1 \text{ lb.}$$

Thus the deflection diagram of the structure when subject to one pound vertical force at a reaction point, is an influence diagram for the vertical reaction at that point.

In the same manner. the deflection diagram of the structure when subject to one pound horizontal force at a reaction point, is an influence diagram for the horizontal reaction at that point.

In the determination of the resisting moment the application of the method is similar. For example: suppose we have given the same truss loaded with a horixontal force of one pound at J. and we wish to know the resisting moment at B caused by this load.

Apply a moment of one inch pound at B (Fig. 19) through a perfectly rigid lever **rigidly** attached to the column at B. Consider this lever as BK of length <u>k</u> inches; then the force at K is 1/k pounds normal to the lever. Find the distance, $d_K + \Delta_4$, between the original position of K and the point where the lever after the force is applied, cuts the horizontal line through K. Find d_J. the horizontal deflection of J. By Maxwell's Theorem, this displacement d_J equals 1/k times what the distance $d_K + \Delta_4$, would be with one pound applied horizontally at J. Now if one pound at J causes the lever to deflect $k \times d_J$ and one inch pound at B causes the lever to deflect $d_K + \Delta_4$, then the moment at B

caused by one pound at J is

$$M_B = \frac{k \times d_J}{d_K + \Delta 4} \times 1 \text{ in. lb.}$$

Thus the deflection diagram of the structure when subject to one inch pound at a reaction point is an influence diagram for the resisting moment at that point.

We shall now use the influence diagram for the determination of the reactions and moments for a typical mill building truss when subject to horizontal loadings. A line diagram of the structure is shown with dimensions in Fig. I. Table I gives the design and general data.

There are two distinct classes to be considered, viz: Class I, columns hinged at their bases; and Class II, columns fixed at their bases. These classes will be considered in the order named.

B. Class I, Columns Hinged at their Bases.

In this case, only the horizontal reactions are statically indeterminate, for there can be no resisting moments at the column bases, and the vertical reactions are easily found by the equations of static equilibrium. We shall determine the influence diagram for the horizontal reaction at B. As has been explained, this diagram is the deflection diagram of the structure when subject to a horizontal force of one pound at B.

In order that the structure, (Fig. 3) may be in equilibrium this horizontal force of one pound at B must be resisted by an

equal and opposite force at D. These horizontal forces at the points of support effect bending in the columns and develop horizontal forces at their points of connection with the truss (Fig. 2). These forces are statically determinate. With respect to the columns they are equal and opposite when referred to the truss.

The final position of the structure is the result of both the flexure of the columns and the deformation of the truss. Fig. 3 shows a grossly exaggerated deflection diagram of the structure under these two equal and opposite forces at the points of support. The point D is considered fixed in position while the point B. free to move horizontally, has moved to B_2. As the conditions of the case presuppose, each column is free to rotate about its base. The outline shown by the solid lines is the initial position and form of the structure; the diagram shown by the broken lines is the initial form of the structure but displaced horizontally through the distance $DD_1 = BB_1$ due to flexure of the column DJ at G. Now consider the truss, deformed by the forces transmitted to it by the columns, taking the position shown by the dotted lines. This deformation together with the flexure of the column BL has moved the point B to its final position B_2. The final position of the structure is shown by the dotted lines.

This deflection diagram can not be drawn to scale until the flexure of the columns and deformation of the truss have been determined. We shall now show the various steps in these

determinations.

Flexure of the Columns.

The forces acting on the two columns are equal but opposite, therefore the flexure of the columns will be the same but opposite.

In supporting the transverse forces at D, G and J (Fig. 2) the columns performs the functions of a beam. Let DG_1J_2 (Fig. 4) represent the elastic curve of the column. The moment diagram is dgj. Draw a tangent to the elastic curve at G_1. The tangential deviations Δ_1, Δ_2, Δ_3 and Δ_4 are found easily by the area moment method.

$$\Delta_1 = \frac{1}{EI}(180 \times 90 \times 120) = \frac{1{,}944{,}000}{EI} = \frac{23{,}940.8867}{E} \text{ inches.}$$

$$\Delta_2 = \frac{1}{EI}(180 \times 36 \times 48) = \frac{311{,}040}{EI} = \frac{3{,}830.5418}{E} \text{ inches.}$$

$$\Delta_3 = \frac{1}{EI}(120 \times 60 \times 30 + 60 \times 40 \times 30) = \frac{288{,}000}{EI} = \frac{3{,}546.7980}{E} \text{ inches.}$$

$$\Delta_4 = \frac{1}{EI}(60 \times 120 \times 60 + 120 \times 60 \times 80) = \frac{1{,}008{,}000}{EI}$$

$$= \frac{12{,}413.7931}{E} \text{ inches.}$$

Whence from Fig. 4,

$$d_{15} = 2\ 1/2\ (d_{11} + \Delta_2) = \frac{15{,}118.671}{E} \text{ inches.}$$

$$d_G = d_{15} + \Delta_1 = \frac{39{,}059.558}{E} \text{ inches.}$$

$$d_J = d_G + d_{11} = \frac{41{,}276.484}{E} \text{ inches.}$$

$$d_F = d_G - (\Delta_3 + 1/3\ d_{15}) = \frac{30{,}473.203}{E} \text{ inches.}$$

$d_E = d_G - (\Delta_4 + 2/3 d_{15})$ $= \frac{16,566.651}{E}$ inches.

Deformation of the Truss.

The forces acting on the truss are shown in Fig. 2. The deformation of the truss may be determined algebraically or graphically.

Algebraic Method. Here use is made of the equation:

$$d = \sum \frac{PUL}{AE}$$ (See footnote)

Where $\underline{d}$ is the displacement in any direction of any panel point of a truss under given loading.

$\underline{P}$ is the stress in the member caused by the given loading.

$\underline{U}$ is the stress in the members caused by unit load placed at the point where the deflection is desired and acting in the direction of the desired displacement.

$\underline{L}$ is the length of the members.

$\underline{A}$ is the cross sectional area of the member.

$\underline{E}$ is the modulus of elasticity of the material.

$\underline{E}$ is constant for all members when the truss is made of the same material throughout. $\underline{P}$ and $\underline{U}$ have positive or negative signs corresponding to tension or compression in the members.

For the history and development of this formula, see article by G. F. Swain in Journal of Franklin Institute, Vol. 85, p. 102: "On the Application of the Principle of Virtual Velocities to the Determination of the Deflection and Stresses of Frames."

The quantity $\frac{PUL}{A}$ is found for each member, and the algebraic sum of these quantities, $\sum \frac{PUL}{A}$, when divided by E gives, d, the desired displacement.

We desire the horizontal displacement of point J(Fig. 3) which we shall designate d_{11}. Then

$$d_{11} = \sum \frac{P_1 U_1 L}{AE}$$

The value of P_1 is the stress in a truss member due to the forces on the truss shown in Fig. 2; value of U_1 is numerically equal to the stress in a truss member due to the forces on the truss shown in Fig. 5; values of L, A and E are as previously explained. Table II is a tabulation of the values of these quantities.

The deformation of the bottom cord JL, which we shall designate $\Delta_1 JL$, is the summation of the strains, $\frac{P_1 L}{A}$, of the bottom cord members. From Table II we find

$$\Delta_1 JL = \frac{4.230 + 5.195 + 3.100 + 5.195 + 4.230}{E} = \frac{930.070}{E} \text{ inches.}$$

$$d_{11} = \sum \frac{P_1 U_1 L}{AE} = \frac{2,216.927}{E} \text{ inches.}$$

Graphic Method: The Williot-Mohr* displacement diagram shown in Fig. 6 furnishes an excellent method for finding the deformation of the truss. Using the strains $\frac{P_1 L}{A}$ given in Table

*For theory and method of construction of the Williot-Mohr diagram see, "Kinetic Theory of Structures", by Molitor.

II, the Williot diagram is drawn assuming the point G fixed in position and the member GJ fixed vertically in direction. The Mohr rotation diagram is then drawn, correcting the error in the assumed direction of GJ. The displacement of any point of the truss is shown in this diagram by the distance from the point on the Mohr diagram, shown in dotted lines, to the corresponding point on the Williot diagram, shown in fine lines. Displacements are up or down and to the right or left according as the point on the Williot diagram is above or below and to the right or left of the corresponding point on the Mohr diagram.

This method, which is really semi-graphical has an advantage over the strictly algebraic method in that it gives at once the displacements of all points on the truss.

The total deflection of B(Fig. 3) is

$$d_B = 2d_G + 2d_{11} + \Delta_1 JL = \frac{83{,}483.039}{E} \text{ inches.}$$

It is now possible to draw to scale the deflection diagram of the structure (Fig. 3) with the horizontal force of one pound at B. Having this diagram we can easily determine the horizontal reactions for loads at various points on the structure.

As has been previously explained; the horizontal reaction at B, due to a load of one pound at any point on the structure, is the deflection, in the direction of the load, of the point loaded, divided by the horizontal deflection of B - both deflections due to one pound horizontal force at B, as in Fig. 3.

The Horizontal Reactions of the right column for loads at various points of the structure follow.

1 lb. horizontal at E: $H_R = \frac{d_E}{d_B} = \frac{16,566.651}{83,483.039} = +0.1984$ lb.

1 lb " " F: $H_R = \frac{d_F}{d_B} = \frac{30,473.203}{83,483.039} = +0.3650$ lb.

1 lb " " G: $H_R = \frac{d_G}{d_B} = \frac{39,059.558}{83,483.039} = +0.4679$ lb.

1 lb " " J: $H_R = \frac{d_J}{d_B} = \frac{41,276.484}{83,483.039} = +0.4944$ lb.

1 lb. normal to roof at 3: $H_R = \frac{d_3}{d_B} = \frac{20,900}{83,483} = +0.2504$ lb.

The horizontal reactions of the left column are statically determined by equating all horizontal forces to zero. The vertical and horizontal reactions of both left and right columns for loads at various points of the structure are given in Table IV, Class I.

C. <u>Class II, Columns Fixed at their Bases.</u>

As previously stated, this class of structures has six unknowns, viz: the horizontal reactions, the vertical reactions and the resisting moments at the points of support. This will necessitate three solutions, one solution for each pair of unknowns.

1. Case A. To Find the Horizontal Reactions.

We shall determine the influence diagram for the horizontal reaction at B. As has been explained, this diagram is the deflection diagram of the structure when subject to a horizontal force of one pound at B.

In order that the structure (Fig. 9) may be in equilibrium this horizontal force of one pound at B must be resisted by an equal and opposite force at D. These horizontal forces at the points of support effect bending in the columns and develop horizontal forces at their points of connection with the truss, (Fig. 7). These forces are not statically determinate on account of the unknown resisting moments at the column bases. For the present these unknown forces will be represented by symbols. Later the numerical values of the forces will be determined by the condition that, at certain points of the structure , the flexure of the columns equals the deformation of the truss. These forces with respect to the columns are equal and opposite when referred to the truss.

The final position of the structure is the result of both the flexure of the columns and the deformation of the truss. Fig. 9 shows a grossly exaggerated deflection diagram of the structure under these two equal and opposite forces at the points of support. The point D is considered fixed in position while the point B, free to move horizontally, has moved to B_2. As the conditions of the cases presuppose, each column must remain vertical at its base. The outline shown in solid lines is the initial position and form of the structure; the diagram shown by the broken lines is the initial form of the structure but displaced horizontally through the distance $DD_1 = BB_1$ due to flexure of the column D J. Now consider the truss, deformed by the forces transmitted to it by the columns, taking the

position shown by the dotted lines. This deformation together with the flexure of the columns BL has moved B to its final position B_2. The final position of the structure is shown by the dotted lines.

This deflection diagram can not be drawn to scale until the flexure of the columns and deformation of the truss have been determined. We shall now show the various steps in these determinations.

Flexure of the Columns.

The forces acting on the two columns are equal but opposite, therefore the flexure of the columns will be the same but opposite.

In supporting the transverse forces at D, G and J, Fig. 7, the column performs the functions of a beam. Let DG_1J_2, (Fig. 8) represent the elastic curve of the column. The moment diagram is dd'gj. The line DJ is tangent to the columns at D. The tangential deviations are easily found by the area-moment method.

$$-d_J = \frac{1}{EI}\left[72A\times36\times48 + 72A\times90\times132 + (72A-180)\times90\times192\right]$$

$$= \frac{1}{EI}(2{,}223{,}936A - 3{,}110{,}400)$$

$$= \frac{27{,}388.3744A - 38{,}305.4187}{E}$$

$$-d_g = \frac{1}{EI}\left[72A\times90\times60 + (72A - 180)\times90\times120\right]$$

$$= \frac{1}{EI}(1{,}116{,}400A - 1{,}944{,}000)$$

$$= \frac{14{,}364.5320A - 23{,}940.8867}{E}$$

$$-d_F = \frac{1}{EI}\left[(72A-60)x60x40 + (72A-180)\ x60x80\right]$$

$$= \frac{1}{EI}\ (518,400A - 1,008,000)$$

$$= \frac{6,384.2365A - 12,413.7931}{E}$$

$$-d_E = \frac{1}{EI}\left[(72A-120)x30x20 + (72A-180)\ x30x40\right]$$

$$= \frac{1}{EI}\ (129,600A - 288,000)$$

$$= \frac{1,596.0591A - 3,546.7980}{E}$$

Deformation of the Truss.

It is now desired to determine the deformation of certain points of the truss under the forces shown in Fig. 7. Until the value of the force A is known, this can be done only by the algebraic method. We wish the values of d_{21}, the horizontal displacement of J; and Δ_2 JL, the deformation of the bottom cord.

$$d_{21} = \sum \frac{P2U_1L}{AE}$$

The value of P_2 is the stress in the truss member due to the forces shown in Fig. 7; value of U_1 is numerically equal to the stress in the member due to the forces shown in Fig. 5. Values of these quantities are tabulated in Table II, from which is also found

$$\Delta_2 JL = \frac{200.9432A + 427.3581}{E}$$

$$d_{21} = \frac{605.5137A + 705.9853}{E}$$

It is now possible to determine the values of the force A. From Fig. 9 may be seen the relation:

$$d_J = d_G + d_{21} \qquad \text{or} \qquad d_J - d_G = d_{21}$$

This gives the equation in A:

$$-13{,}023.8424A + 14{,}364.5320 = 605.5137A + 705.9853$$

Whence

$$A = + 1.0021$$

This value of A is now substituted in the expressions for the flexure of the columns and the deformations of the truss and the numerical values of these quantities determined.

$d_J = \frac{10.858.3784}{E}$ inches. $d_G = \frac{9{,}545.5859}{E}$ inches.

$d_F = \frac{6{,}015.8816}{E}$ inches. $d_E = \frac{1{,}947.3201}{E}$ inches.

The last column in Table II gives the strains, $\frac{P_2L}{A}$, of the truss members, from which can be drawn the Williot-Mohr diagram as shown in Fig. 10. Again the Williot diagram is drawn by assuming the point G fixed in position and the members G J fixed in direction. The Mohr diagram is then drawn correcting the error in the assumed direction of GJ. Displacements are up or down and to the right or left according as the point on the Williot diagram is above or below and to the right or left of the point on the Mohr diagram.

The total deflection of B. Fig. 9, is

$$d_B = 2d_G + 2d_{11} + \Delta_2 JL = \frac{22{,}345.4955}{E} \text{ inches}$$

It is now possible to draw to scale the deflection diagram of the structure (Fig. 9) caused by the horizontal forces of one pound at B and D. Having this diagram, we can easily determine the horizontal reactions for loads at various points on the structure.

[illegible]

These reactions are:

1 lb. horizontal at E: $H_R = \frac{1,947.3201}{22,345.4955} = 0.0871$ lbs.

1 " " " F: $H_R = \frac{6,015.8816}{22,345.4955} = 0.2692$ lbs.

1 " " " G: $H_R = \frac{9,545.5859}{22,345.4955} = 0.4272$ lbs.

1 " " " J: $H_R = \frac{10,858.3784}{22,345.4955} = 0.4859$ lbs.

1 " normal at 3: $H_R = \frac{6,310}{22,345.4955} = 0.2824$ lbs.

The horizontal reactions of the left column are found by equating all horizontal forms to zero. The horizontal reactions of both columns for the loads at the various points are tabulated in Table IV, Class II.

2. Case B, To Find the Vertical Reactions.

We shall determine the influence diagram for the vertical reaction at B. As has been explained, this diagram is the deflection diagram of the structure when subject to a vertical force of one pound at B.

In order that the structure (Fig. 13) may be in equilibrium this vertical force of one pound at B must be resisted by an equal and opposite force at D, and couples at the column bases. These vertical forces at the points of support effect bending in the columns and develop horizontal forces at their points of connection with the truss, (Fig. 11). These forces are statically indeterminate on account of the unknown couples at the column bases. For the present, these unknown forces will

be represented by symbols. Later the numerical values of the forces will be determined by the condition that, at certain points of the structure, the flexure of the column equals the deformation of the truss. These forces with respect to the columns are equal and opposite when referred to the truss.

The final position of the structure is the result of both the flexure of the columns and the deformation of the truss. Fig. 13 shows a grossly exaggerated deflection diagram of the structure under these two equal and opposite forces and the resisting couples at the points of support. The point D is considered fixed in position while the point B, free to deflect vertically has moved to B_2. The resisting couples cause each column to remain vertical at its base. The diagram shown in solid lines is the initial position and form of the structure; the diagram shown by the broken lines is the initial form of the structure but displaced horizontally through the distance $DD_1 = BB_1$ due to flexure of the columns D J. Now consider the truss deformed by the forces transmitted to it by the columns, to take the position shown by the dotted line. This deformation together with the flexure of the column BL has moved B to its final position B_2. The final position of the structure is shown by the dotted lines.

This deflection diagram cannot be drawn to scale until the flexure of the columns and deformation of the truss have been determined. We shall now show the various steps in these determinations.

Flexure of the Columns.

In this case the forces acting on the two columns are not the same, so the columns will be considered separately.

Column DJ: Here the forces are the same as in the preceeding case, (Fig. 8) except that which was the numerical force in the other case now has the coefficient C.

$$-d_J = \frac{27,388.3744A - 38,305.4187C}{E}$$

$$-d_G = \frac{14,364.5320A - 23,940.8867C}{E}$$

$$-d_F = \frac{6,384.2365A - 12,413.7931C}{E}$$

$$-d_E = \frac{1,596.0591A - 3,546.7980}{E}$$

Column BL: In Fig. 12 let $B_2K_2L_2$ represent the elastic curve of the column. The moment diagram is bb'kl. The line B_2L_1 is tangent to the column at B_2. The tangential deviations are easily found by the area-moment method.

$$d_L = \frac{1}{EI}\left[(72A+480)x(36x48+180x162) - 180C \times 90 \times 192\right]$$

$$= \frac{1}{EI}(2,223,936A - 3,110,400C + 14,826,240)$$

$$= \frac{27,388.3744A - 38,305.4187C + 182,589.1630}{E}$$

$$d_K = \frac{1}{EI}\left[(72A + 480) \times 180 \times 90 - 180C \times 90 \times 120\right]$$

$$= \frac{1}{EI}(1,166,400A - 1,944,000C + 7,776,000)$$

$$= \frac{14,364.5320A - 23,940.8867C + 95,763.5468}{E}$$

Deformation of the Truss.

It is now desired to determine the displacement of certain points of the truss under the forces shown in Fig. 11. Until the value of the forces A and C are known, this can be done only by the algebraic method. We wish the values of d_{33}, the horizontal displacement of K; and $\Delta_3 JL$, the deformation of the bottom cord.

$$d_{33} = \sum \frac{P_3 U_3 L}{AE}$$

The value of P_3 is the stress in the truss member due to the forces shown in Fig. 11; value of U_3 is the numerical value of the stress in the truss member due to the forces shown in Fig. 14. Values of these quantities are tabulated in Table III, from which is also found

$$d_{33} = 1,411.9714A + 1,839.3294C + 4,706.5954$$

$$\Delta_3 JL = \frac{200.9432A + 427.3581C + 670.0936}{E}$$

It is now possible to determine the values of the forces A and C. From Fig. 13 we see the relations:

$d_K = d_G + d_{33}$ or $d_K - d_G - d_{33} = 0$

$d_L = d_J + \Delta_3 JL$ or $d_L - d_J - \Delta_3 JL = 0$

Which gives the equations:

$$27,317.0926A - 49,721.1028C + 91,056.9514 = 0$$

$$54,575.8056A - 77,038.1955C + 181,919.0694 = 0$$

Whence

$A = -3.3333$ $\qquad C = 0$

These values are significant. It was found that in the

twenty trusses analyzed the value of C is zero and A has the negative value of one half the numerical horizontal force on the right column. This numerical force equals one pound times the span length divided by the distance from top of column to foot of knee brace. Therefore the value of A is independent of all characteristics of the truss other than the two mentioned above.

These values of A and C are now substituted in the expressions for the flexure of the columns and the deformation of the truss to get the numerical values of these quantities.

$d_J = \frac{91,294.5812}{E}$ inches. $d_G = \frac{47,881.7733}{E}$ inches.

$d_F = \frac{21,280.7883}{E}$ inches. $d_E = \frac{5,320.1970}{E}$ inches.

The twelfth column of Table III gives the strains $\frac{P_3L}{A}$, of the truss members, from which can be drawn the Williot diagram shown in Fig. 16. This diagram is somewhat different from the preceeding displacement diagrams. Here use is made of the fact that we know the relative positions of G and J. G is assumed fixed in position and the point J is located the distance $d_J - d_G$ to the right, and the distance equal to the elongation of GJ, above G. With the diagram drawn in this manner the displacements of all points are represented by their distances from G. Displacements are up or down and to the right or left according as the point is above or below and to the right or left of G.

The vertical deflection d_{35} of the truss at K is given by the formula,

$$d35 = \sum \frac{P_3U_5L}{A}$$

The values of $\frac{P_3 L}{A}$ are the strains in the truss members due to the loads shown in Fig. 11; value of U_5 is numerically equal to the stress in the members due to the forces shown in Fig. 15. Values of these quantities are given in Table III.

$$d_{35} = \frac{7,480.7510}{E} \text{ inches.}$$

The total vertical deflection of B,(Fig. 13)is: 6 2/3(d_J - d_G). that due to the rotation of the truss about G_1. plus d_{35}. the vertical deformation of the truss at K_2.

$$d_B = 6\ 2/3(d_J - d_G) + d_{35} = \frac{296,899.4705}{E} \text{ inches.}$$

It is now possible to draw to scale the deflection diagram of the structure (Fig. 13) with the vertical force of one pound at B. Having this diagram, we can easily determine the vertical reactions for loads at various points on the structure.

These reactions are:

1 lb. horizontal at E:	$V_R = \frac{5,320.1970}{296,899.4705} = 0.0179$ lbs.
1 " " " F:	$V_R = \frac{21,280.7883}{296,899.4705} = 0.0717$ lbs.
1 " " " G:	$V_R = \frac{47,881.7733}{296,899.4705} = 0.1613$ lbs.
1 " " " J:	$V_R = \frac{91,294.5812}{296,899.4705} = 0.3075$ lbs.
1 " normal"to roof at 3:	$V_R = \frac{123,000}{296,899.4705} = 0.4143$ lbs.

The vertical reactions of the left column are found by equating all vertical forms to zero. The vertical reactions for both columns for loads at the various points on the structure are tabulated in Table IV, Class II.

3. Case C, To Find the Resisting Moments.

We shall determine the influence diagram for the resisting moment at B. As has been explained, this diagram is the deflection diagram of the structure when subject to a moment of one inch-pound at B.

In order that the structure (Fig. 19) may be in equilibrium, the bending moment of one inch pound at B is resisted by horizontal and vertical forces at the points of support and a resisting moment at the point of support of the left column. These forces and moments at the supports effect bending in the columns and develop horizontal forces at their points of connection with the truss, (Fig. 17). These forces are statically indeterminate, and for the present these unknown forces will be represented by symbols. Later their numerical values will be determined by the condition that, at certain points of the structure, the flexure of the columns equals the deformation of the truss. These forces with respect to the columns are equal and opposite when referred to the truss.

The final position of the structure is the result of both the flexure of the columns and the deformation of the truss. Fig. 19 shows a grossly exaggerated deflection diagram of the structure under the applied bending moment and the resultant resisting forces and moment. The points D and B are considered fixed in position. The left column is fixed in direction at its base while the right column may take any direction. The outline shown in solid lines is the initial position and form

of the structure; the diagram shown in broken lines is the initial form of the structure but displaced horizontally through the distance $DD_1 = BB_1$ due to flexure of the column DJ. Now consider the truss, deformed by the forces transmitted to it by the columns, to take the position shown by the dotted lines. This deformation together with the flexure of the column BL has moved the structure to its final position shown by the dotted lines.

This deflection diagram cannot be drawn to scale until the flexure of the columns and deformation of the truss have been determined. We shall now show the various steps in these determinations.

Flexure of the Columns.

In this case the forces acting on the two columns are not the same, so the columns will be considered separately.

Column DJ: Here the forces are the same as in the preceeding case,(Fig. 11). Therefore the deflections will be the same.

$$-d_J = \frac{27,388.3744A - 38,305.4187C}{E}$$

$$-d_G = \frac{14,364.5320A - 23,940.8867C}{E}$$

$$-d_F = \frac{6,384.2365A - 12,413.7931C}{E}$$

$$-d_E = \frac{1,596.0591A - 3,546.7980C}{E}$$

Column BL: In Fig. 18, let BK_2L2 represent the elastic curve of the column. The moment diagram is bb'kl. The line TU is tangent to the column at K_2; and the line BK', through which the moment at B is applied, is tangent to the column at B. The tangential deviations are easily found by the area-moment method.

$$\Delta_1 = \frac{1}{EI}\left[(1-180C)x90x120 + 1x90x60\right]$$

$$= \frac{1}{EI}\,(16,200 - 1,944,000C)$$

$$= \frac{199.5074 - 23,940.8867C}{E}$$

$$-\Delta_2 = \frac{1}{EI}\left[(1-180C) \times 36 \times 48\right]$$

$$= \frac{1}{EI}\,(1728 - 311,040C)$$

$$= \underline{21.2808 - 3,830.5419C}$$

$$\Delta_4 = \frac{1}{EI}\left[(1-180C)\ x90 \times 60 + 1 \times 90 \times 120\right]$$

$$= \frac{1}{EI}\,(16,200 - 972,000C)$$

$$= \frac{199.5074 - 11,970.4434C}{E}$$

$$\Delta_5 = \frac{1}{EI}\left[(1-180C) \times (36x48+90x132) + 1 \times 90 \times 192\right]$$

$$= \frac{1}{EI}(\ 30,888 - 2,449,440C)$$

$$= \frac{380.3941 - 30,165.5172C}{E}$$

Deformation of the Truss.

It is now desired to determine the displacement of certain points of the truss under the forces shown in Fig. 17. Until the values of A, C, and R are known, this can be done only by the algebraic method. We wish the value of d_{41}, the horizontal displacement of J; d_{43}, the horizontal displacement of K; and $\Delta_4 JL$ the deformation of the bottom cord.

$$d_{41} = \sum \frac{P_4 U_1 L}{AE} \qquad d_{43} = \sum \frac{P_4 U_3 L}{AE} \qquad \Delta_4 JL = \sum \frac{P_4 L}{AE}$$

The value of P_4 is the stress in the truss member due to the forces shown in Fig. 17: value of U_1 is numerically equal to the stress in the truss members due to the forces shown in Fig. 5: and the value of U_3 is numerically equal to the stress in the truss members due to the forces shown in Fig. 14. Forces for P_4, Fig. 17, are similar to forces for P_3, Fig. 11, which greatly facilitates the computations. Values of these different quantities are given in Table III.

$$d_{41} = \frac{605.5137A + 705.9853 + 865.8960R}{E}$$

$$d_{43} = \frac{1411.9714A + 1839.3294C + 4706.5954R}{E}$$

$$4JL = \frac{200.9432A + 427.3581C + 670.0936R}{E}$$

It is now possible to determine the values of forces A, C and R, by the following relations seen from Fig. 19.

$$d_J = d_G + d_{41} \qquad \text{or} \qquad d_J - d_G - d_{41} = 0 \qquad (1)$$

$$d_K = d_G + d_{43} \qquad \text{or} \qquad d_K - d_G - d_{43} = 0 \qquad (2)$$

and the equation of R in terms of A and C seen from Fig. 19.

$$252A + 1680R - 180A - 180C - 1200R + 1 = 0$$

$$72A + 480R - 180C + 1 = 0$$

$$R = \frac{-72A + 180C - 1}{480} \qquad (3)$$

We have not the expression for the value of d_K but will express this quantity in terms of known quantities, (Fig. 18.)

$$d_K = 5/7(d_L + \Delta_5) - \Delta_4$$

$$7d_K = 5d_L + 5\Delta_5 - 7\Delta_4$$

But

$$d_L = d_J + \Delta_4 JL$$

Therefore

$$7d_K = 5d_J + 5\Delta_4 JL + 5\Delta_5 - 7\Delta_4$$

Substituting this in (1) we get:

$$5(d_J + \Delta_4 JL + \Delta_5) - 7(d_G + d_{43} + \Delta_4) = 0 \qquad (2a)$$

Eliminating R by use of equations (3) we get the following equations from equations (1) and (2a)

$$+ 13{,}499.4717A - 13{,}333.8357C - 1.8040 = 0$$

$$+ 40{,}829.8767A + 64{,}930.4990C - 367.0765 = 0$$

Solving for A and C then substituting in equation (3) we get

$A = + 0.005404 \qquad C = + 0.005336 \qquad R = - 0.0008932$

These values of A, C and R are now substituted in the expressions for the flexure of the columns and the deformation of the truss to get the numerical values of these quantities.

$d_J = \frac{56.3917}{E}$ inches. $d_G = \frac{50.1162}{E}$ inches.

$d_F = \frac{31.7360}{E}$ inches. $d_E = \frac{10.2995}{E}$ inches.

The last column of Table III gives the strains, $\frac{P_4L}{A}$, of the truss members, from which can be drawn the Williot-Mohr diagram shown in Fig. 20. Here the point G is assumed fixed in position and the members GJ fixed in direction. The Mohr rotation diagram is then drawn, correcting the error in the assumed direction of GJ. The displacement of any point of the truss is shown on the diagram by the distance from the point on the Mohr diagram, shown in dotted lines, to the corresponding point on the Williot diagram, shown in fine lines. Displacements are up or down and to the right or left according as the point on the Williot diagram is above or below and to the right or left of the corresponding point on the Mohr diagram.

By substituting the values of <u>A</u>, <u>C</u> and <u>R</u> in the expressions for the following deformations we get:

$d_{41} = \frac{6.2661}{E}$ $\Delta_1 = \frac{71.7588}{E}$

$d_{43} = \frac{13.2423}{E}$ $\Delta_2 = \frac{-0.8410}{E}$

$\Delta_{4JL} = \frac{2.7679}{E}$ $\Delta_4 = \frac{135.6331}{E}$

The deflection of the tangent is,

$$d_K + \Delta_4 = d_G + d_{43} + \Delta_4 = \frac{198.9916}{E} \text{ inches.}$$

It is now possible to draw to scale the deflection diagram of the structure, Fig. 19, with the bending moment of <u>one inch pound</u> at B. Having this diagram, we can easily determine

the resisting moments at the right column base for loads at various points on the structure.

These resisting moments are:

1 lb. horizontal at E: $M_R = \frac{180 \times 10.30}{198.99} = 9.318$ in. lbs.

1 " " " F: $M_R = \frac{180 \times 31.74}{198.99} = 28.711$ in. lbs.

1 " " " G: $M_R \; \frac{180 \times 50.12}{198.99} = 45.337$ in. lbs.

1 " " " J: $M_R \; \frac{180 \times 56.39}{198.99} = 51.009$ in. lbs.

1 " normal to roof at 3: $M_R \; \frac{180 \times 31.96}{198.99} = 28.910$ in. lbs.

The resisting moment at the left column bases is now easily found by statics. The resisting moments of both columns for loads at various points of the structure are given in Table IV Class II.

III.

APPLICATION TO DETERMINATION OF WIND REACTIONS.

A. Method.

While there is some difference of opinion as to the action of wind on a structure such as a mill building, the conventional method of treatment considers the windward side of the building subject to a uniform horizontal load, and the windward slope of the roof subject to a uniform normal load. This normal load is a function of the horizontal load and the pitch of the roof.

The intensity of the horizontal load to be considered will depend upon local conditions. The three formulas used for computing the normal load are given and discussed in "Steel Mill Buildings", by Ketchum. The writer quotes from this book, Third Edition, page 14.

"The normal component of the wind pressure on inclined surfaces has usually been computed by Hutton's empirical formula

$$P_n = P \sin A^{1.842 \cos A - 1}$$

where P_n equals the normal component of the wind pressure, P equals the pressure per square foot of vertical surface; and A equals the angle of inclination of the surface with the horizontal.

The formula due to Duchemin

$$P_n = P \frac{2 \sin A}{1 + \sin^2 A}$$

Where P_n, P and A are the same as above, gives results considerably larger for ordinary roofs than Hutton's formula, and is coming into quite general use.

The formula (Straight Line Formula)

$$P_n = \frac{P}{45} A$$

Where P_n and P are the same as above, and A is the angle of inclination of the surface in degrees (A being equal to or less than 45 degrees), gives results which agree very closely with

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Hutton's formula, and is much more simple.

Hutton's formula is based on experiments which were very crude and probably erroneous. Duchemin's formula is based on very careful experiments and is now considered the most reliable formula in use. The Straight Line formula agrees with experiments quite closely and is preferred by many engineers on account of its simplicity."

In order that the results may be general and that the empirical formulas may be easily applied in all cases, the writer has used as the value of P, <u>one pound per linear foot of column.</u> The value of P_n used, is in accordance with Duchemin's formula. For the one-quarter pitch roof, this value of P_n is:

$$P_n = \frac{2 \sin A}{1 + \sin^2 A} = \frac{2 \times 0.4472}{1+ (.4472 \times 4472} = 0.7455 \text{ lbs. per lin ft}$$

We shall now determine the reactions of the structure when subject to a horizontal load of one pound per linear foot on the windward column, and a normal load of 0.7455 pounds per linear foot on the windward slope of the roof. The reactions for <u>any</u> loading are directly proportioned to the reactions for this unit load. In all cases the wind will be considered as coming from the left and the reactions of the right or leeward column will be first determined.

The wind loads are considered to be applied at the girts on the column and at the purlins on the roof. The girt points are E, F, G and J; and the purlin points are J, 2, 3, 6 and 7. The load at each point is one half the load between that point and the adjacent point on either side. The method of determining the reactions and resisting moments is as follows:

Determine the displacement of the load points in the direction of the load, and the displacement of the reaction point in the direction of the desired reaction; in the case of the resisting moments, determine the displacement of the lever. These displacements are those due to a unit load or unit moment at the reaction point. Determine the wind loads at each load point.

The horizontal or vertical reaction at B then is:

$$R = \frac{\sum Pd}{d_B}$$

where P is the load at each point; d is the deflection of the point in the direction of the load; and d_B is the displacement of the reaction point B in the direction of the desired reaction.

In the determination of the resisting moment at B the application is similar.

$$M_B = \frac{k \sum Pd}{d_K + \Delta_4}$$

where P and d are the same as above; k is the length of the lever by which the unit moment at B is applied; and $d_K + \Delta_4$ is the deflection of the lever.

The reactions of the windward columns may then be determined by the principles of static equilibrium.

B. Trusses Analyzed.

This thesis presents the results of the analysis of twenty mill building trusses. Line diagram of the structures drawn to the same scale with dimensions are shown in Fig. 21. All have

Fink trusses with one quarter pitch roof. The trusses of 30, 40, 50 and 60 foot span were designed on the assumption that the bents were placed 15 feet apart. The truss of twenty foot span was made much heavier than necessary to see what effect this might have on the results. The truss design is constant for each span length. The columns of the 30, 40, 50 and 60 foot spans with the 16, 21 and 26 foot heights have the same section. This column section corresponds to the design for the 40 foot span and 21 foot column structure, considering the bents placed 15 feet apart and assuming the point of contraflexure at one-third the height to the knee brace. The column sections for the 31 foot height are all the same and somewhat heavier than that required. The columns of the 20 foot span are constant in section and likewise heavier than required.

These structures were chosen primarily to study the effect of variations in span length, column height and the ratio of column height to span length. Time has not permitted a study of the effect of variations in all the characteristics of the structure, such as pitch and type of roof, etc.

Line diagrams of one structure of each span length, are given in Figs. 22 to 26. The designs of the structures are given in Tables V to IX.

C. <u>Results.</u>

Each of the twenty structures was analyzed by the method which has been presented, but only the results of the analyses

are given herewith.

The stresses and corresponding strains of the truss members for each case are given complete in Table X to XIX for one truss of each span length. For the other trusses, only the resulting strains are given in Tables XX to XXIV. Only one influence diagram of each span length for each reaction will be given, since these diagrams are not essential to the results. The Williot-Mohr displacement diagram of each truss for each reaction is given, since these diagrams are essential. The vertical reaction is an exception to the above statements for it was found that the internal deformation of the truss was negligible and the results could be more readily obtained by the construction of the influence diagram.

The displacements of the points on the columns, including the reaction point and the lever in the case of the resisting moment, were computed, while the displacements of points on the truss were obtained wholly or in part by graphics. Points on the truss are affected by both the flexure of the left column and the deformation of the truss. The former effect in the line of direction of the normal load equals the displacement d_G times the sine of the angle of inclination of the roof. The latter effect is found by scaling from the displacement diagram of the truss. For example, the displacement normal to the roof of the point 3, due to deformation of the truss, is represented by the distance parallel to the roof, between points 3 on the Mohr and Williot diagrams. These distances are scaled and their values

given on the diagrams.

The results are summarized and given in tabular form on pages 48 to 97.

In making a comparison of the results for different structures it was first necessary to decide upon a comparison factor; a quantity which shows the true effect of variations in different characteristics of the structure. After much study the best comparison factor found was the reaction divided by the total height of the structure. This factor furnishes an excellent means for comparing the results for the variations considered, and it is believed that it will render as good service in the study of the effect of variations of any other characteristic of the structure. The comparison factor may be called the reaction factor for when multiplied by the total height of the structure the reaction is obtained. Values of this quantity are given with the results on pages 48 to 97 as H/t, V/t, and M/t, and in Table XXV. Where t is the total height of the structure.

The point of contraflexure of each column can be located when the horizontal reactions and resisting moments are determined. Denoting by Y the height of the point of contraflexure above the base of the columns, we have:

For the leeward column:

$$M_Y = H_R \times Y - M_R = 0 \quad \text{or } Y_R = \frac{M_R}{H_R}$$

For the windward column;

$$M_Y = H_L \times Y - M_L - \frac{wY^2}{2} = 0$$

When $\underline{w}$ is the load on the column, 1/12 lb. per inch.

$$Y_L^2 - 24H_L Y + 24\ M_L = 0$$

$$Y_L = +\ 12H_L \pm \sqrt{144H_L^2 - 24M_L}$$

Values of Y_L and Y_R together with Y_L/d and Y_R/d, where $\underline{d}$ is the height to the foot of the knee brace, are given with the results on pages 98 to 97 and in Table XXV.

The effect of variations in column height, span length and ratio-column height to span length on each reaction factor and on the location of the point of contraflexure is shown in Graphs No. 1 to 26. These graphs will be discussed later.

The writer thought at first of presenting algebraic formulas for the reactions but it was found that such formulas are so complicated that the reactions could best be obtained directly from the graphs. An example is given to show how the graphs are used.

Let us find the reactions, position of the points of contraflexure, and position and amount of maximum bending moment of each column in a mill building bent of the dimensions and loading given below.

Bents which are spaced 16 ft. on centres, have 36 ft. span, 20 ft columns and Fink one-quarter pitch roof trusses. Building is subject to a wind loading of 20 lbs per square foot of vertical surface.

As it was found that, within practical limits, the design

of the columns and truss members and position of knee brace have no appreciable effect on the reactions, the only characteristics of the bents in which we are interested are height of columns and length of span.

We shall find the reactions for both hinged and fixed columns.

Span 36 ft. - Col. height 20 ft.

Ratio column height to span length, $h/l = \frac{20}{36} = 0.556$

Total height of structure, $t = 20 + 1/4 \times 36 = 29$ ft.

Load per vertical linear foot of column = $20 \times 16 = 320$ lbs.

Class I, Hinged Columns.

Case A; Horizontal Reactions. Graph 5.

Leeward column reaction, $H_R = 0.350 \times 29 \times 320 = 3,250$ lbs.

Windward column reaction, $H_L = 0.570 \times 29 \times 320 = \underline{5,290 \text{ lbs.}}$

$H = 8,540$ lbs.

Ratio $\frac{H_R}{H} = 0.380$

Case B; Vertical Reactions. Graphs 12 and 13.

Leeward column reaction $V_R = 0.463 \times 29 \times 320 = 4,300$ lbs.

Windward column reaction $V_L = 0 \times 29 \times 320 = \underline{0}$

$V = 4,300$ lbs.

Class II, Fixed Columns.

Case A; Horizontal Reactions. Graph 6.

Leeward Columnsreaction, $H_R = 0.316 \times 29 \times 320 = 2,930$ lbs.

Windward column reaction, $H_L = 0.604 \times 29 \times 320 = \underline{5,610 \text{ lbs.}}$

$H = 8,540$ lbs.

Ratio $\frac{H_R}{H} = 0.343$

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Case B; Vertical Reactions. Graphs 12 and 13.

Leeward column reaction $V_R = 0.295 \times 29 \times 320 = 2,740$ lbs.

Windward column reaction, $V_L = 0.170 \times 29 \times 320 = \underline{1,550 \text{ lbs.}}$

$V = 4,300$ lbs.

Case C; Resisting Moments. Graphs 14 and 15.

Leeward column moment, $M_R = 32.5 \times 29 \times 320 = 301,600$ in. lbs.

Windward column moment, $M_L = 40.0 \times 29 \times 320 = 371,200$ in. lbs

While the points of contraflexure might be located from the graphs, they can be more accurately located by means of algebraic equations involving H and M.

$$Y_R = \frac{M_R}{H_R} = \frac{301,600}{2,930} = 102.9 \text{ in.} \qquad \frac{Y_R}{d} = \frac{102.9}{180} = 0.572$$

$$Y_L = \frac{+2H_L \pm \sqrt{4H_L^2 - 8\,w\,M_L}}{2w} \qquad w = \frac{320}{12} = 26.67 \text{ lbs. per in.}$$

$$= \frac{2 \times 5610 \pm \sqrt{125,888,400 - 79,189,342}}{2 \times 26.67}$$

$$= 82.3 \text{ in.} \qquad \frac{Y_L}{d} = \frac{82.3}{180} = 0.457$$

These positions of points of contraflexure are consistent with those which would be found from Graphs 23 and 24.

The Maximum Moment in each column will occur either at the base of the column or at the foot of the knee brace. Denoting the moment at the knee brace in the leeward column by M_K, and the moment at the knee brace in the windward column by M_G; we have:

$$M_K = (180 - 102.9) \times 2930 = 225,900 \text{ in. lbs.}$$

$$M_G = (180 - 82.3) \times 5610 - \frac{26.67 \times 97.7 \times 97.7}{2}$$

$$= 548,100 - 254,500 = 293,600 \text{ in. lbs.}$$

Moments:	Windward Col.	Leeward Col:
At knee brace:	293,600 in. lbs.	225,900 in. lbs.
At Column base:	371,200 in. lbs.	301,600 in. lbs.

IV

DISCUSSION AND CONCLUSIONS.

In the discussions which follow, it must be borne in mind that the statements made apply only to mill-building bents with one-quarter pitch Fink roof trusses. The main purpose of the thesis has been to determine the effect of variations in column height and span length. Incidentally, however, the effects of design of column sections and truss members, and the position of knee brace have been determined. More time could now be most profitably spent on the investigation of the effects of type and pitch of roof truss.

A. Horizontal Reactions.

Graphs 1 and 2 show the Reaction Factors for hinged and fixed columns plotted with column heights as abscissas. The Reaction Factors for each span length have an individual line of variation. These lines are very nearly parallel and consistently spaced, showing that variations in the other characteristics of the structure of this type and roof-pitch have but little effect on the results.

Graphs 3 and 4 show the Reaction Factors plotted with span lengths as abscissas. Here the individual lines of variations for the different column heights are parallel and consistently spaced.

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Graphs 5 and 6 show the Reaction Factors and the ratio H_R/H plotted with the ratios h/l, (column-height to span-length) as abscissas. Here the individual lines of variation for span length and column height approximately coincide - the dispersion being greater in the case of the fixed columns. These graphs show that the ratio h/l is the factor which more nearly determines the ratio H_R/H. This ratio H_R/H decreases as the ratio h/l increases. Within the limits given the variations are from 0.44 to 0.34 for hinged columns; and from 0.45 to 0.29 for fixed columns.

Graph 7, which shows graphs 5 and 6 drawn on the same sheet, gives a good comparison of the results for hinged and fixed columns.

Conclusions:

Column height and span length are the only characteristics of the structure which materially effect the horizontal reactions. The ratio h/l is the factor which more nearly determines the ratio of the reactions.

The degree of fixity of the columns bases has only small effect on the horizontal reactions, the ratio H_R/H being in general slightly greater for the hinged columns.

For the usual range of ratios of column height to span length, from 0.45 to 0.65, the ratio H_R/H is about 0.38 and 0.35 for the hinged and fixed columns, respectively. The common assumption of equal horizontal reactions is quite in error.

B. Vertical Reactions.

Graphs 8 and 9 show the Reaction Factors for hinged and fixed columns plotted with column heights as abscissas. The Reaction Factors for each span length have an individual line of variation. These lines are approximately parallel and consistently spaced, showing that variations in the other characteristics of the structure have but little effect on the results. The Reaction Factor of the leeward column for fixed columns is practically constant.

Graphs 10 and 11 show the Reaction Factors plotted with span lengths as abscissas. Here the individual lines of variation for the different column heights are consistently spaced.

Graphs 12 and 13 show the Reaction Factors plotted with ratio h/l as abscissas. Here the individual lines of variation for span lengths and column heights coincide, showing that the ratio h/l is the factor which more nearly determines the Vertical Reaction Factor.

Conclusions:

Column height and span length are the only characteristics of the structure which appreciably affect the vertical reactions. The ratio h/l is the factor which more nearly determines the ratio of the reactions.

The degree of fixity of the column bases greatly affects the vertical reactions. As the columns are more rigidly fixed the vertical reactions of the leeward columns decreases and that of the windward column correspondingly increases.

The vertical reaction of the windward column decreases as the ratio h/l increases and in the case of the hinged columns reaches a large negative value.

C. Resisting Moments.

Graphs 14 and 15 show the Reaction Factors plotted with column heights as abscissas. Here the individual lines of variation for the span lengths approximately coincide. This shows that the column height is the factor which most greatly influences the resisting moment.

Graphs 16 and 17 show the Reaction Factors plotted with spam lengths as abscissas. Here the individual lines of variation for each column height are approximately parallel and consistently spaced, showing that variations in the other characteristics of the structure have but little effect on the results. The fact that the lines are approximately horizontal indicates that the span length has but little effect on the moments.

Graphs 18 and 19 show the Reaction Factors plotted with the ratio h/l as abscissas.

Graph 20, which shows graphs 14 and 15 drawn on the same sheet, gives a good comparison of the moments of the leeward and windward columns. These are the best graphs from which to make empirical determinations of the resisting moments.

Graphs 21 to 26 show the variation of the points of contraflexure with column height, span length and ratio h/l . In all these graphs the ordinates are Y/d(the ratio of height of point

[illegible]

of contraflexure to height of foot of knee brace.) These graphs show:

(a) leeward column - the ratio y/d is practically constant, being only slightly affected by variations in any characteristics of the structure.

(b) windward column - here the position of the knee brace seems to have a material effect. This is seen from Graph 22 which shows that the ratio y/d is practically the same for those trusses which have the same ratio of supported to unsupported length of column, $\frac{h-d}{d}$. For the ordinary values of $\frac{h-d}{d}$ the ratio y/d is fairly constant but when this ratio approaches the value 0.5 the ratio y/d increases very rapidly. On account of the effect of position of knee brace the points which represent the same column heights on graphs 23 and 24 cannot be connected by smooth curves.

Conclusions:

Column height is the only characteristic of the structure which materially influences the resisting moments.

An increase in span length slightly increases the resisting moment of the leeward column and decreases that of the windward column.

Contrary to what might be expected, the moment of inertia of the columns and the position of the knee brace do not seem to affect the resisting moments.

The location of the point of contraflexure of the leeward

column is practically constant at the ratio Y/d equal to 0.60.

The location of the point of contraflexure of the windward is practically constant at the ratio Y/d equal to 0.50 until the ratio of supported to unsupported lengths of columns approaches 0.50, then the ratio Y/d increases very rapidly.

While the common assumption of the location of the points of contraflexure are not greatly in error, the resulting bending moments are greatly in error for the reason that the assumption of equal horizontal reactions is far from correct.

The position and amount of the maximum bending moment in the columns had best be determined by an analysis of the structure or by use of the graphs as given herein and not by making arbitrary assumptions.

Span 20 ft - Col. height 10 ft

Total height of structure $t = 10 + 5 = 15$ ft.
Normal load on roof segment $5.5902 \times 0.7455 = 4.167$ lbs.

Class I.

Case A:
Total Horizontal load, $H = 10x1 + 5x0.7455 = 13.728$ lbs.

$d_B = 6{,}241$ $d_G \sin A = 2{,}421 \times 0.4472 = 1{,}083$

See Fig. 28

$d_J = 3{,}010$ $d_J^1 = 1{,}083 + 275 = 1358$
$d_G = 2{,}421$ $d_O = 1{,}083 + 975 = 2058$
$d_F = 1{,}383$ $d_S = 1{,}083 + 887 = 1970$

J: $3{,}010 \times 2 = 6020$ J: $1358 \times 2.084 = 2830$
G: $2{,}421 \times 3\tfrac{1}{2} = 8474$ O: $2058 \times 4.167 = 8576$
F: $1{,}383 \times 3 = 4149$ S: $1970 \times 2.084 = 4105$
18643 15511

$18{,}643 + 15{,}511 = 34{,}154$

$H_R = \dfrac{34{,}154}{6{,}241} = 5.473$ lbs. $H_L = 13.728 - 5.473 = 8.255$ lbs.

$\dfrac{H_R}{t} = \dfrac{5.473}{15} = 0.365$ $\dfrac{H_L}{t} = \dfrac{8.255}{15} = 0.550$ $\dfrac{H_R}{H} = \dfrac{5.473}{13.728} = 0.399$

Case B:
Total Vertical reaction, $V = 10 \times 0.7455 = 7.455$ lbs.

$V_R = \dfrac{10x5 + 0.7455x11.180x9.95}{20} = \dfrac{50 + 82.930}{20} = 6.646$ lbs.

$\dfrac{V_R}{t} = 0.443$ $V_L = V - V_R = 0.809$ lbs. $\dfrac{V_L}{t} = 0.0539$

Class II

Case A:
$d_B = 1{,}972$ $d_G \sin A = 564 \times 0.4472 = 252$

See Fig. 33.

$d_J = 902$	$d_J^1 = 252 + 157 = 409$
$d_G = 564$	$d_O = 252 + 590 = 842$
$d_F = 199$	$d_S = 252 + 550 = 802$

J:	$902 \times 2 = 1804$		J:	$409 \times 2.084 =$	852
G:	$564 \times 3\frac{1}{2} = 1974$		Φ:	$842 \times 4.167 =$	3509
F:	$199 \times 3 = \underline{597}$		S:	$802 \times 2.084 =$	$\underline{1671}$
	4375				6032

$$4375 + 6032 = 10{,}407$$

$$H_R = \frac{10{,}407}{1{,}972} = 5.278 \text{ lbs.} \qquad H_L = 13.728 - 5.278 = 8.450 \text{ lbs.}$$

$$\frac{H_R}{t} = 0.352 \qquad \frac{H_L}{t} = 0.563 \qquad \frac{H_R}{H} = 0.385$$

Case B:

$d_B = 20{,}374$

See Fig. 37

$d_J = 6060$	$d_J^1 = 2700$
$d_G = 2300$	$d_O = 8500$
$d_F = 576$	$d_S = 14200$

J:	$6060 \times 2 = 12{,}120$		J:	$2700 \times 2.084 =$	5,630
G:	$2300 \times 3\frac{1}{2} = 8{,}060$		O:	$8500 \times 4.167 =$	35,420
F:	$576 \times 3 = \underline{1{,}730}$		S:	$14200 \times 2.084 =$	$\underline{29{,}590}$
	21,910				70,640

$$21{,}910 + 70{,}640 = 92{,}550$$

$$V_R = \frac{92{,}550}{20{,}374} = 4.543 \text{ lbs.} \qquad V_L = 7.455 - 4.543 = 2.912 \text{ lbs.}$$

$$\frac{V_R}{t} = 0.303 \qquad \frac{V_L}{t} = 0.194$$

Case C.

$d_K + \Delta_4 = 25.55$ $\qquad d_G \sin A = 5.141 \times 0.4472 = 2.299$

See Fig. 43.

$d_J = 7.944$	$d_J^1 = 2.299 + 1.290 = 3.589$
$d_G = 5.141$	$d_Φ = 2.299 + 4.50 = 6.799$
$d_F = 1.838$	$d_S = 2.299 + 3.90 = 6.199$

J: 7.944 x 2 = 15.888
G: 5.141 x $3\frac{1}{2}$ = 17.993
F: 1.838 x 3 = 5.514
39.395

J: 3.589 x 2.084 = 7.479
O: 6.799 x 4.167 = 28.331
S: 6.199 x 2.084 = 12.919
48.729

39.395 + 48.729 = 88.124

$$M_R = \frac{72 \times 88.124}{25.552} = \frac{6,344.928}{25.552} = 248.31 \text{ in. lbs.}$$

$$M_L = \sum Pd - 240V_R - M_R$$

$$= (10x5x12)+(0.7455x11.180x9.95x12)-(240x4.543)-248.31$$

$$= 600 + 995.16 - 1090.32 - 248.31 = 256.53 \text{ in. lbs.}$$

$$\frac{M_R}{t} = 16.55 \qquad \frac{M_L}{t} = 17.10$$

Points of Contraflexure:

$$Y_R = \frac{M_R}{H_R} = \frac{248.31}{5.278} = 47.05 \text{ in.} \qquad Y_L = 12H_L \pm \sqrt{144H^2 - 24M_L} = 37.3 \text{ in.}$$

$$\frac{Y}{d} = \frac{47.05}{72} = 0.654 \qquad \frac{Y_L}{d} = \frac{37.3}{72} = 0.518$$

Span 20 ft. - Col. height 12 ft.

Total height of structure $t = 12 + 5 = 17$ ft.
Normal load on roof segment 5.5902 x 0.7455 = 4.167 lbs.

Class I

Case A: (Fig. 27)
Total Horizontal load H = 12x1+5x0.7455 = 15.728 lbs.

d_B = 11,015 $d_G \sin A$ = 4683 x 0.4472 = 2094

J:	5385 x 2 = 10,770		J:	2424 x 2.084 = 5,052
G:	4683 x 4 = 18,732		O:	3254 x 4.167 = 13,559
F:	2751 x 4 = 11,004		S:	3139 x 2.084 = 6,542
	40,506			25,153

See Fig. 29

$$40{,}506 + 25{,}153 = 65{,}659$$

$$H_R = \frac{65{,}659}{11{,}015} = 5.961 \text{ lbs.} \qquad H_L = H - H_R = 9.767 \text{ lbs.}$$

$$\frac{H_R}{t} = 0.351 \qquad \frac{H_L}{t} = 0.575 \qquad \frac{H_R}{H} = 0.379$$

Case B:

Total Vertical reaction $V = 10 \times 0.7455 = 7.455$ lbs.

$$V_R = \frac{12\text{x}6 + 0.7455\text{x}11.180\text{x}10.85}{20} = \frac{72 + 90.431}{20} = 8.122 \text{ lbs.}$$

$$\frac{V_R}{t} = 0.478 \qquad V_L = V - V_R = -0.667 \text{ lbs.} \qquad \frac{V_L}{t} = -0.0393$$

Class II

Case A: (Fig. 32)

$d_B = 3{,}215$ $\qquad d_G \sin A = 1120 \times 0.4472 = 501$

See Fig. 34.

J:	1517 x 2 = 3034		J:	684 x 2.084 = 1425
G:	1120 x 4 = 4480		O:	1181 x 4.167 = 4921
F:	416 x 4 = 1664		S:	1131 x 2.084 = 2357
	9178			8703

$$9178 + 8703 = 17{,}881$$

$$H_R = \frac{17{,}881}{3{,}215} = 5.562 \text{ lbs.} \qquad H_L = H - H_R = 10.166 \text{ lbs.}$$

$$\frac{H_R}{t} = 0.327 \qquad \frac{H_L}{t} = 0.598 \qquad \frac{H_R}{H} = 0.354$$

Case B:

$d_B = 25{,}494$

See Fig. 38 and 41

J: 8875 x 2 = 17750
G: 4096 x 4 = 16384
F: 1024 x 4 = 4096
38230

J: 4000 x 2.084 = 8340
Φ: 10900 x 4.167 = 45420
S: 18300 x 2.084 = 38140
91900

38,230 + 91,900 = 130,130

$V_R = \frac{130,130}{25,494} = 5.104$ lbs. $V_L = 7.455 - 5.104 = 2.351$ lbs.

$\frac{V_R}{t} = 0.300$ $\frac{V_L}{t} = 0.138$

Case C: (Fig. 42)

$d_K + \Delta_4 = 39.820$ $d_G \sin A = 4.027$
See Fig. 44

J: 11.898 x 2 = 23.796
G: 9.005 x 4 = 36.020
F: 3.393 x 4 = 13.572
73.388

J: 5.357 x 2.084 = 11.164
O: 8,667 x 4.167 = 36.115
S: 8.027 x 2.084 = 16.728
64.007

73.388 + 64.007 = 137.395

$$M_R = \frac{96 \times 137.395}{39.820} = \frac{13,189.920}{39.820} = 331.24 \text{ in. lbs.}$$

$$M_L = \sum Pd - 240V_R - M_R$$

$$= (12x6x12)+(0.7455x11.180x10.85x12)-(240x5.104)-331.24$$

$$= 864 + 1085.17 - 1224.96 - 331.24 = 382.97 \text{ in. lbs.}$$

$\frac{M_R}{t} = 19.48$ $\frac{M_L}{t} = 22.53$

Points of Contraflexure:

$Y_R = \frac{M_R}{H_R} = \frac{331.24}{5.562} = 59.6$ in. $Y_L = 12H_L \pm \sqrt{144H_L^2 - 24M_L} = 46.6$

$\frac{Y}{d} = \frac{59.6}{96} = 0.620$ $\frac{Y}{d} = \frac{46.6}{96} = 0.486$

Span 20 ft. - Col. height 16 ft.

Total height of structure $t = 16 + 5 = 21$ ft.
Normal load on roof segment $5.5902 \times 0.7455 = 4.167$ lbs.

Class I

Case A:
Total Horizontal load $H = 16x1 + 5 \times 0.7455 = 19.728$ lbs

$d_B = 27,387$ $d_G \sin A = 12,618 \times 0.4472 = 5643$

See Fig. 30

J:	$13,548 \times 2 = 27,096$	J:	$6068 \times 2.084 = 12,646$
G:	$12,618 \times 5 = 63,090$	O:	$7153 \times 4.167 = 29,807$
F:	$7,691 \times 6 = 46,146$	S:	$6993 \times 2.084 = 14,573$
	$136,332$		$57,026$

$136,332 + 57,026 = 193,358$

$H_R = \frac{193,358}{27,387} = 7.060$ lbs. $H_L = H - H_R = 12.668$ lbs.

$\frac{H_R}{t} = 0.336$ $\frac{H_L}{t} = 0.603$ $\frac{H_R}{H} = 0.358$

Case B:
Total Vertical reaction $V = 10 \times 0.7455 = 7.455$ lbs.

$V_R = \frac{16x8 + 0.7455x11.180x12.650}{20} = \frac{128 + 105.434}{20} = 11.672$ lbs.

$\frac{V_R}{t} = 0.556$ $V_L = V - V_R = -4.217$ lbs. $\frac{V_L}{t} = -0.201$

Class II

Case A:
$d_B = 7410$ $d_G \sin A = 3089 \times 0.4472 = 1381$

See Fig. 35.

$$\begin{array}{lrcl} J: & 3602 \times 2 & = & 7204 \\ G: & 3089 \times 5 & = & 15445 \\ F: & 1233 \times 6 & = & \underline{7398} \\ & & & 30047 \end{array} \qquad \begin{array}{lrcl} J: & 1616 \times 2.084 & = & 3368 \\ O: & 2231 \times 4.167 & = & 9297 \\ S: & 2156 \times 2.084 & = & \underline{4493} \\ & & & 17158 \end{array}$$

$$30047 + 17158 = 47{,}205$$

$$H_R = \frac{47{,}205}{7410} = 6.370 \text{ lbs.} \qquad H_L = H - H_R = 13.358 \text{ lbs.}$$

$$\frac{H_R}{t} = 0.303 \qquad \frac{H_L}{t} = 0.636 \qquad \frac{H_R}{H} = 0.323$$

Case B:

$d_B = 35{,}734$ See Fig. 39

$$\begin{array}{lrcl} J: & 16043 \times 2 & = & 32086 \\ G: & 9216 \times 5 & = & 46080 \\ F: & 2304 \times 6 & = & \underline{13824} \\ & & & 91990 \end{array} \qquad \begin{array}{lrcl} J: & 7200 \times 2.084 & = & 15000 \\ O: & 17500 \times 4.167 & = & 72920 \\ S: & 27500 \times 2.084 & = & \underline{57310} \\ & & & 145230 \end{array}$$

$$91{,}990 + 145{,}230 = 237{,}220$$

$$V_R = \frac{237{,}220}{35{,}734} = 6.639 \text{ lbs.} \qquad V_L = V - V_R = 0.816 \text{ lbs.}$$

$$\frac{V_R}{t} = 0.316 \qquad \frac{V_L}{t} = 0.0389$$

Case C:

$d_K + \Delta_4 = 78.133$ $d_G \sin A = 8.676$

See Fig. 45.

$$\begin{array}{lrcl} J: & 22.305 \times 2 & = & 44.610 \\ G: & 19.402 \times 5 & = & 97.010 \\ F: & 7.828 \times 6 & = & \underline{46.968} \\ & & & 188.588 \end{array} \qquad \begin{array}{lrcl} J: & 10.006 \times 2.084 & = & 20.853 \\ O: & 13.236 \times 4.167 & = & 55.154 \\ S: & 12.626 \times 2.084 & = & \underline{26.313} \\ & & & 102.320 \end{array}$$

$$188.588 + 102.320 = 290.908$$

$$M_R = \frac{144 \times 290.908}{78.133} = \frac{41{,}890.752}{78.133} = 536.15 \text{ in.lbs.}$$

$$M_L = \sum Pd - 240V_R - M_R$$

$$= (16\text{x}8\text{x}12) + (0.7455\text{x}11.180\text{x}12.65\text{x}12) - (240\text{x}6.639) - 536.15$$

$$= 1536 + 1265.21 - 1593.36 - 536.15 = 671.70 \text{ in lbs.}$$

$$\frac{M_R}{t} = 25.53 \qquad \frac{M_L}{t} = 31.99$$

Points of Contraflexure:

$Y_R = \frac{M_R}{H_R} = \frac{536.15}{6.370} = 84.2$ in. $\quad Y_L = 12H_L \pm \sqrt{144H_L^2 - 24M_L}$
$= 62.6$ in.

$\frac{Y}{d} = \frac{84.2}{144} = 0.585 \qquad \frac{Y}{d} = \frac{62.6}{144} = 0.435$

Span 20 ft. - Col. height 19 ft.

Total height of structure $\quad t = 19 + 5 = 24$ ft.
Normal load on roof segment $\quad 5.5902 \times 0.7455 = 4.167$ lbs.

Class I

Case A:
Total Horizontal load $\quad H = 19x1 + 5x0.7455 = 22.728$ lbs.

$d_B = 71{,}419 \qquad d_G \sin A = 15404$

See Fig. 31.

J:	35545 x 2 = 71090	J:	15904 x 2.084 = 33144
G:	34446 x 4½ = 155007	O:	17184 x 4.167 = 71606
F:	27398 x 5 = 136990	S:	16994 x 2.084 = 35415
E:	15029 x 5 = 75145		
	438232		140165

438,232 + 140,165 = 578,397

$H_R = \frac{578{,}397}{71{,}419} = 8.099$ lbs. $\qquad H_L = H - H_R = 14.629$ lbs.

$\frac{H_R}{t} = 0.337 \qquad \frac{H_L}{t} = 0.610 \qquad \frac{H_R}{H} = 0.356$

Case B:
Total Vertical load $\quad V = 10 \times 0.7455 = 7.455$ lbs.

$V_R = \frac{19x9.5+0.7455x11.180x14.00}{20} = \frac{180.5 + 116.686}{20} = 14.859$ lbs.

$\frac{V_R}{t} = 0.619 \qquad V_L = V - V_R = -7.404$ lbs $\qquad \frac{V_L}{t} = -0.308$

Class II.

Case A:

$d_B = 12,450$ $\qquad$ $d_G \sin A = 5514 \times 0.4472 = 2466$

See Fig. 36

J:	6114 x 2 = 12,228		J:	2741 x 2.084 =	5,712
G:	5514 x $4\frac{1}{2}$ = 24,813		O:	3456 x 4.167 =	14,401
F:	3517 x 5 = 17,585		S:	3366 x 2.084 =	7,015
E:	1146 x 5 = 5,730				
	60,356				27,128

$$60,356 + 27,128 = 87,484$$

$$H_R = \frac{87,484}{12,450} = 7.027 \text{ lbs.} \quad H_L = H - H_R = 15.701$$

$$\frac{H_R}{t} = 0.293 \qquad \frac{H_L}{t} = 0.654 \qquad \frac{H_R}{H} = 0.309$$

Case B:

$d_B = 43,414$

See Fig. 40.

J:	22,763 x 2 = 45,530		J:	10,050 x 2.084 =	20,940
G:	14,400 x $4\frac{1}{2}$ = 64,800		O:	22,500 x 4.167 =	93,760
F:	6,400 x 5 = 32,000		S:	35,000 x 2.084 =	72,940
E:	1,600 x 5 = 8,000				
	150,330				187,640

$$150,330 + 187,640 = 337,970$$

$$V_R = \frac{337,970}{43,414} = 7.785 \text{ lbs.} \qquad V_L = V - V_R = -0.330 \text{ lbs.}$$

$$\frac{V_R}{t} = 0.324 \qquad \frac{V_L}{t} = -0.0138$$

Case C:

$d_K + \Delta_4 = 115.687$ $\qquad$ $d_G \sin A = 13.173$

See Fig. 46

J:	32.326 x 2 = 64.652		J:	14.483 x 2.084 =	30.183
G:	29.456 x $4\frac{1}{2}$ = 132.552		O:	17.673 x 4.167 =	73.643
F:	18.926 x 5 = 94.630		S:	17.073 x 2.084 =	35.580
E:	6.190 x 5 = 30.950				
	322.784				139.406

$$322.784 + 139.406 = 462.190$$

$$M_R = \frac{180 \times 462.190}{115.687} = \frac{83,194}{115.687} = 719.13 \text{ in. lbs.}$$

$$M_L = \sum Pd - 240V_R - M_R$$

$$= (19 \times 9.5 \times 12) + (0.7455 \times 11.180 \times 14.00 \times 12) - 240 \times 7.785 - 719.13$$

$$= 2166 + 1400.23 - 1868.40 - 719.13 = 978.70 \text{ in. lbs.}$$

$$\frac{M_R}{t} = 29.96 \qquad \frac{M_L}{t} = 40.78$$

Points of Contraflexure:

$$Y_R = \frac{M_R}{H_R} = \frac{719.13}{7.027} = 102.3 \text{ in.} \qquad Y_L = 12H_L \pm \sqrt{144H_L^2 - 24M} = 79.1 \text{ in.}$$

$$\frac{Y}{d} = \frac{102.3}{180} = 0.569 \qquad \frac{Y}{d} = \frac{79.1}{180} = 0.439$$

Span 30 ft. - Col height 16 ft.

Total height of structure $t = 16 + 7.5 = 23.5$ ft.
Normal load on roof segment $4.1927 \times 0.7455 = 3.126$ lbs.

Class I

Case A: (Fig. 47)
Total Horizontal load $H = 16x1+7.5x0.7455 = 21.591$ lbs

$d_B = 45,914$ $d_G \sin A = 21,048 \times 0.4472 = 9,413$

See Fig. 48.

J:	$22,616 \times 2 = 45,232$	J: $10,143 \times 1.563 = 15,854$
G:	$21,048 \times 4 = 84,192$	2: $11,613 \times 3.126 = 36,302$
F:	$16,302 \times 4 = 65,208$	3: $12,063 \times 3.126 = 37,709$
E:	$8,832 \times 4 = 35,328$	6: $12,313 \times 3.126 = 38,490$
	$229,960$	7: $11,983 \times 1.563 = 18,729$
		$147,084$

$229,960 + 147,084 = 377,044$

$H_R = \frac{377,044}{45,914} = 8.212$ lbs. $H_L = H - H_R = 13.379$ lbs.

$\frac{H_R}{t} = 0.349$ $\frac{H_L}{t} = 0.569$ $\frac{H_R}{H} = 0.380$

Case B:
Total Vertical load $V = 15 \times 0.7455 = 11.182$ lbs.

$V_R = \frac{16x8+0.7455x16.771x15.55}{30} = \frac{128+194.418}{30} = 10.747$ lbs.

$\frac{V_R}{t} = 0.457$ $VL = V - V_R = 0.435$ lbs. $\frac{V_L}{t} = 0.0185$

Class II

Case A:
$d_B = 12,530$ $d_G \sin A = 5165 \times 0.4472 = 2310$

See Fig. 53

J: 6040 x 2 = 12,080
G: 5165 x 4 = 20,660
F: 3204 x 4 = 12,816
E: 1028 x 4 = 4,112
49,668

J: 2725 x 1.563 = 4,259
2: 3580 x 3.126 = 11,191
3: 3830 x 3.126 = 11,973
6: 3970 x 3.126 = 12,410
7: 3790 x 1.563 = 5,924
45,757

49,668 + 45,757 = 95,425

$$H_R = \frac{95,425}{12.530} = 7.616 \text{ lbs.} \qquad H_L = H - H_R = 13.975 \text{ lbs.}$$

$$\frac{H_R}{t} = 0.324 \qquad \frac{H_L}{t} = 0.595 \qquad \frac{H_R}{H} = 0.353$$

Case B:
d_B = 133,714

See Fig. 57 and 61

J: 40,008 x 2 = 80,020
G: 22,983 x 4 = 91.930
F: 10,215 x 4 = 40,860
E: 2,554 x 4 = 10,220
223,030

J: 18,500 x 1.563 ≛ 28,900
2: 36,500 x 3.126 = 114,100
3: 55,000 x 3.126 = 171,900
6: 73,500 x 3.126 = 229,800
7: 91,500 x 1.563 = 143,000
687,700

223,030 + 687,700 = 910,730

$$V_R = \frac{910,730}{133,714} = 6.811 \text{ lbs.} \qquad V_L = V - V_R = 4.371 \text{ lbs.}$$

$$\frac{V_R}{t} = 0.290 \qquad \frac{V_L}{t} = 0.186$$

Case C: (Fig. 62)
$d_K + \Delta_4 = 130.225$ $\qquad d_G \sin A = 14.655$

See Fig. 63

J: 37.807 x 2 = 75.614
G: 32.771 x 4 = 131.084
F: 20.484 x 4 = 81.936
E: 6.601 x 4 = 26.404
315.038

J: 17.035 x 1.563 = 26.626
2: 21.805 x 3.126 = 68.162
3: 22.855 x 3.126 = 71.445
6: 23.200 x 3.126 = 72.523
7: 21.800 x 1.563 = 34.073
272.829

315.038 + 272.829 = 587.867

$$M_R = \frac{144 \text{x} 587.867}{130.225} = \frac{84,652.848}{130.225} = 650.05 \text{ in. lbs.}$$

$M_L = \sum Pd - 360V_R - M_R$

$= (16x8x12) + (0.7455x16.771x15.55x12) - (360x6.811) - 650.05$

$= 1536 + 2333.02 - 2451.96 - 650.05 = 767.01$ in. lbs.

$\frac{M_R}{t} = 27.66$ $\qquad\qquad$ $\frac{M_L}{t} = 32.64$

Points of Contraflexure:

$Y_R = \frac{M_R}{H_R} = \frac{650.05}{7.616} = 85.4$ in. $\qquad$ $Y_L = 12H_L \pm \sqrt{144H_L^2 - 24M_L} = 69.3$ in.

$\frac{Y}{d} = \frac{85.4}{144} = 0.593$ $\qquad\qquad$ $\frac{Y}{d} = \frac{69.3}{144} = 0.481$

Span 30 ft. - Col. height 21 ft.

Total height of structure $t = 21 + 7.5 = 28.5$ ft.
Normal load on roof segment $4.1927 \times 0.7455 = 3.126$ lbs.

Class I.

Case A:
Total Horizontal load $H = 21x1 + 7.5x0.7455 = 26.591$ lbs.

$d_B = 108,344$ $\qquad$ $d_G \sin A = 51,713 \times 0.4472 = 23,130$

See Fig. 49.

J:	53,751 x 2	= 107,500		J:	24,050 x 1.563	= 37,590
G:	51,713 x 4	= 206,850		2:	25,980 x 3.126	= 81,210
F:	45,078 x 4	= 180,310		3:	26,480 x 3.126	= 82,780
E:	34,017 x 4	-= 136,070		6:	26,730 x 3.126	= 83,560
Q:	19,891 x 4½	= 89,510		7:	26,310 x 1.563	= 41,120
		720,240				326,260

$720,240 + 326,260 = 1,046,500$

$H_R = \frac{1,046,500}{108,344} = 9.659$ lbs. $\quad H_L = H - H_R = 16.932$ lbs.

$\frac{H_R}{t} = 0.339$ $\qquad$ $\frac{H_L}{t} = 0.594$ $\qquad$ $\frac{H_R}{H} = 0.364$

Case B:

Total Vertical load $V = 15 \times 0.7455 = 11.182$ lbs.

$$V_R = \frac{21 \times 10.5 + 0.7455 \times 16.771 \times 17.80}{30} = \frac{220.5 + 222.550}{30} = 14.768 \text{ lbs.}$$

$\frac{V_R}{t} = 0.518$ $V_L = V - V_R = -3.586$ $\frac{V_L}{t} = -0.126$

Class II.

Case A:

$d_B = 28{,}372$ $d_G \sin A = 12{,}808 \times 0.4472 = 5{,}728$

J:	13,922	x 2	=	27,844		J;	6,238	x	1.563	=	9,750
G:	12,808	x 4	=	51,232		E:	7,338	x	3.126	=	22,939
F:	9,888	x 4	=	39,552		3:	7,628	x	3.126	=	23,845
E:	5,888	x 4	=	23,552		6:	7,753	x	3.126	=	24,236
Q:	2,172	x 4½	=	9,774		7:	7,498	x	1.563	=	11,719
				151,954							92,489

See Fig. 54.

$151{,}954 + 92{,}489 = 244{,}443$

$H_R = \frac{244{,}443}{28{,}372} = 8.616$ lbs. $H_L = H - H_R = 17.975$ lbs.

$\frac{H_R}{t} = 0.302$ $\frac{H_L}{t} = 0.631$ $\frac{H_R}{H} = 0.324$

Case B:

$d_B = 181{,}595$

See Fig. 58

J:	69,535	x 2	=	139,070		J:	31,600	x	1.563	=	49,400
G:	46,126	x 4	=	184,500		2:	57,000	x	3.126	=	178,200
F:	26,973	x 4	=	107,890		3:	83,000	x	3.126	=	259,500
E:	12,928	x 4	=	51,710		6:	109,000	x	3.126	=	340,700
Q:	3,990	x 4½	=	17,960		7:	134,000	x	1.563	=	209,400
				501,130							1,037,200

$501{,}130 + 1{,}037{,}200 = 1{,}538{,}330$

$V_R = \frac{1{,}538{,}330}{181{,}505} = 8.471$ lbs. $V_L = V - V_R = 2.711$ lbs.

$\frac{V_R}{t} = 0.297$ $\frac{V_L}{t} = 0.0951$

Case C:

$d_K + \Delta_4 = 241.020$ $d_G \sin A = 28.076$

See Fig. 64.

J:	67.696 x 2 = 135.392	J:	30.326 x 1.563 = 47.400
G:	62.782 x 4 = 251.128	E:	34.976 x 3.126 = 109.335
F:	48.709 x 4 = 194.836	3:	35.926 x 3.126 = 112.305
E:	29.095 x 4 = 116.380	6:	36.276 x 3.126 = 113.399
Q:	10.754 x $4\frac{1}{2}$ = 48.393	7:	34.976 x 1.563 = 54.667
	746,129		437.106

746.129 + 437.106 = 1183.235

$$M_R = \frac{204 \times 1183.235}{241.020} = \frac{241,379.940}{241.020} = 1001.49 \text{ in. lbs.}$$

$$M_L = \sum Pd - 360V_R - M_R$$

$$= (21 \text{x} 10.5 \text{x} 12) + (0.7455 \text{x} 16.771 \text{x} 17.80 \text{x} 12) - (360 \text{x} 8.471) - 1001.493$$

$$= 2646 + 2670.60 - 3049.56 - 1001.49 = 1265.55 \text{ in. lbs.}$$

$\frac{M_R}{t} = 35.14$ $\qquad$ $\frac{M_L}{t} = 44.41$

Points of Contraflexure:

$Y_R = \frac{M_R}{H_R} = \frac{1001.49}{8.616} = 116.24$ in. $\qquad$ $Y_L = 12H_L \pm \sqrt{144^2_H - 24M_L} = 88.8$ in.

$\frac{Y}{d} = \frac{116.24}{204} = 0.570$ $\qquad$ $\frac{Y}{d} = \frac{88.8}{204} = 0.435$

Span 30 ft. - Col. height 26 ft.

Total height of structure	t= 26 + 7.5 = 33.5 ft.
Normal load on roof segment	4.1927 x 0.7455 = 3.126 lbs.

Class I.

Case A:

Total Horizontal load $\quad H = 26 \text{x} 1 + 7.5 \times 0.7455 = 31.591$ lbs

$d_B = 212,139$ $\qquad$ $d_G \sin A = 103,060 \times 0.4472 = 46,088$

See Fig. 50.

J:	105,569	x	2	=	211,138
G:	103,060	x	4	=	412,240
F:	94,537	x	4	=	378,148
E:	79,885	x	4	=	319,540
Q:	60,465	x	$4\frac{1}{2}$	=	272,092
S:	31,563	x	5	=	157,815
					1,750,973

J:	47,238	x	1.563	=	73,833
E:	49,588	x	3.126	=	155,012
3:	50,288	x	3.126	=	157,200
6:	50,638	x	3.126	=	158,294
7:	50,108	x	1.563	=	78,319
					622,658

1,750,973 + 622,658 = 2,373,631

$H_R = \frac{2,373,631}{212,139} = 11.189$ lbs. $\quad H_L = H - H_R = 20.402$ lbs.

$\frac{H_R}{t} = 0.334 \qquad \frac{H_L}{t} = 0.609 \qquad \frac{H_R}{H} = 0.354$

Case B:

Total Vertical load $V = 15 \times 0.7455 = 11.182$ lbs.

$V_R = \frac{26\text{x}13 + 0.7455\text{x}16.771\text{x}20.05}{30} = \frac{338 + 250.681}{30} = 19.623$ lbs.

$\frac{V_R}{t} = 0.586 \qquad V_L = V - V_R = -8.441$ lbs. $\qquad \frac{V_L}{t} = -0.252$

Case Class II

Case A:

$d_B = 54,557 \qquad d_G \sin A = 25623 \times 0.4472 = 11,459$

J:	26,974	x	2	=	53,948
G:	25,623	x	4	=	102,492
F:	21,750	x	4	=	87,000
E:	15,938	x	4	=	63,752
Q:	9,550	x	$4\frac{1}{2}$	=	42,975
S:	2,831	x	5	=	14,155
					364,322
					364,322

See Fig. 55.

J;	12,079	x	1.563	=	18,879
E:	13,399	x	3.126	=	41,885
3:	13,739	x	3.126	=	42,948
6:	13,909	x	3.126	=	43,480
7:	13,624	x	1.563	=	21,294
					168,486

364,322 + 168,486 = 532,808

$H_R = \frac{532,808}{54,557} = 9.766$ lbs. $\quad H_L = H - H_R = 21.825$ lbs.

$\frac{H_R}{t} = 0.292 \qquad \frac{H_L}{t} = 0.652 \qquad \frac{H_R}{H} = 0.309$

Case B:

$d_B = 229{,}477$ See Fig. 59.

J:	107,042	x	2	=	214,080
G:	77,249	x	4	=	309,000
F:	51,712	x	4	=	206,850
E:	31,283	x	4	=	125,130
Q:	15,961	x	$4\frac{1}{2}$	=	71,820
S:	3,990	x	5	=	19,950
					946,830

J:	48,000	x	1.563	=	75,000
E:	80,000	x	3.126	=	250,100
3:	113,000	x	3.126	=	353,200
6:	147,000	x	3.126	=	459,500
7:	177,000	x	1.563	=	276,600
					1,414,400

$$946{,}830 + 1{,}414{,}400 = 2{,}361{,}230$$

$$V_R = \frac{2{,}361{,}230}{229{,}477} = 10.290 \text{ lbs.} \qquad V_L = V - V_R = 0.902 \text{ lbs.}$$

$$\frac{V_R}{t} = 0.307 \qquad \frac{V_L}{t} = 0.0269$$

Case C:

$d_K + \Delta_4 = 388.494$ $d_G \sin A = 44.882$

See Fig. 65.

J:	105.078	x	2	=	210.156
G:	100.363	x	4	-=	401.452
F:	85.500	x	4	=	342.000
E:	62.802	x	4	=	251.208
Q:	37.695	x	$4\frac{1}{2}$	=	169.627
S:	11.190	x	5	=	55.950
					1,430.393

J:	47.082	x	1.563	=	73.589
E:	51.562	x	3.126	=	161.183
3:	52.442	x	3.126	=	163.933
6:	52.732	x	3.126	=	164.840
7:	51.262	x	1.563	=	80.123
					643.668

$$1{,}430.393 + 643.668 = 2{,}074.061$$

$$M_R = \frac{264 \times 2{,}074.061}{388.494} = \frac{547{,}552.104}{388.494} = 1409.42 \text{ in. lbs.}$$

$$M_L = \sum Pd - 360V_R - M_R$$

$$= (26\times13\times12) + (0.7455\times16.771\times20.05\times12) - (360\times10.290) - 1490.42$$

$$= 4056 + 3008.17 - 3704.40 - 1409.42 = 1950.35 \text{ in. lbs.}$$

$$\frac{M_R}{t} = 42.07 \qquad \frac{M_L}{t} = 58.22$$

Points of Contraflexure:

$$Y_R = \frac{M_R}{H_R} = \frac{1409.42}{9.766} = 144.3 \text{ in.} \qquad Y_L = 12H \pm \sqrt{144H_L^2 - 24M_L}$$

$$= 114.0 \text{ in.}$$

$$\frac{Y}{d} = \frac{144.32}{264} = 0.547 \qquad \frac{Y}{d} = \frac{114.0}{264} = 0.432$$

Span 30 ft. - Col. height 31 ft.

Total height of structure. $t = 31 + 7.5 = 38.5$ ft.
Normal load on roof segment. $4.1927 \times 0.7455 = 3.126$ lbs.

Class I

Case A:
Total horizontal load $H = 31 + 7.5 \times 0.7455 = 36.591$ lbs

$d_B = 164,493$ $d_G \sin A = 78,694 \times 0.4472 = 35,192$
See Fig. 51.

J:	81,665 x 2 = 163,330		J:	36,532 x 1.563 = 57,100
G:	78,694 x 4 = 314,776		2:	39,342 x 3.126 = 122,983
F:	73,003 x 4 = 292,012		3:	40,092 x 3.126 = 125,328
E:	64,448 x 4 = 257,792		6:	40,472 x 3.126 = 126,515
Q:	53,526 x 4½ = 240,867		7:	39,742 x 1.563 = 62,117
S:	37,306 x 5 = 186,530			494,043
N:	19,139 x 5 = 95,695			
	1,551,002			

$1,551,002 + 494,043 = 2,045,045$

$H_R = \frac{2,045,045}{164,493} = 12.432$ lbs. $H_L = H - H_R = 24.159$ lbs.

$\frac{H_R}{t} = 0.323$ $\frac{H_L}{t} = 0.628$ $\frac{H_R}{H} = 0.340$

Case B:
Total Vertical load $V = 15 \times 0.7455 = 11.182$ lbs.

$V_R = \frac{31 \times 15.5 + 0.7455 \times 16.771 \times 22.30}{30} = \frac{480.5 + 278.812}{30} = 25.310$ lbs.

$\frac{V_R}{t} = 0.657$ $V_L = V - V_R = -14.128$ lbs. $\frac{V_L}{t} = -0.367$

Class II.

Case A:
$d_B = 42,340$ $d_G \sin A = 8,655$
See Fig. 56.

J:	20,842 x 2 = 41,684		J:	9,325 x 1.563 =	14,575
G:	19,354 x 4 = 77,416		E:	10,785 x 3.126 =	33,714
F:	16,790 x 4 = 67,160		3:	11,160 x 3.126 =	34,886
E:	13,321 x 4 = 53,284		6:	11,355 x 3.126 =	35,496
Q:	9,476 x $4\frac{1}{2}$ = 42,642		7:	11,025 x 1.563 =	17,232
3:	4,860 x 5 = 24,300				135,903
N:	1,377 x 5 = 6,885				
	313,371				

313,371 + 135,903 = 449,274

$H_R = \frac{449,274}{42,340} = 10.611$ lbs. $H_L = H - H_R = 25.980$ lbs.

$\frac{H_R}{t} = 0.276$ $\frac{H_L}{t} = 0.675$ $\frac{H_R}{H} = 0.290$

Case B:

$d_B = 105,261$ See Fig. 60

J:	55,790 x 2 = 111,580		J:	25,000 x 1.563 =	39,100
G:	42,558 x 4 = 170,230		2:	39,600 x 3.126 =	123,800
F:	30,882 x 4 = 123,530		3:	54,500 x 3.126 =	170,400
E:	21,074 x 4 = 84,300		6:	69,000 x 3.126 =	215,700
Q:	13,135 x $4\frac{1}{2}$ = 59,110		7:	84,000 x 1.563 =	131,300
S:	5,838 x 5 = 29,190				680,300
N:	1,459 x 5 = 7,290				
	585,230				

585,230 + 680,300 = 1,265,530

$V_R = \frac{1,265,530}{105,261} = 12.022$ lbs. $V_L = V - V_R = -0.840$ lbs.

$\frac{V_R}{t} = 0.312$ $\frac{V_L}{t} = -0.0218$

Case C:

$d_K + \Delta_4 = 222.822$ $d_G \sin A = 25.772$

See Fig. 66.

J:	61,582 x 2 = 123.164		J:	27.602 x 1.563 =	43.142
G:	57.630 x 4 = 230.520		E:	31.272 x 3.126 =	97.756
F:	50.283 x 4 = 201.132		3:	32.022 x 3.126 =	100.101
E:	40.090 x 4 = 160.360		6:	32.222 x 3.126 =	100.726
Q:	28.587 x $4\frac{1}{2}$ = 128.641		7:	31.092 x 1.563 =	48.597
S:	14.705 x 5 = 73.525				390.322
N:	4.176 x 5 = 20.880				
	938.222				

938.222 + 390.322 = 1,328.544

$$M_R = \frac{324 \times 1328.544}{222.822} = \frac{430,448.256}{222.822} = 1931.80 \text{ in. lbs.}$$

$$M_L = \sum Pd - 360V_R - M_R$$

$$= (31x15.5x12) + (0.7455x16.771x22.30x12) - (360x12.022) - 1931.80$$

$$= 5766 + 3345.74 - 4327.92 - 1931.80 = 2852.02 \text{ in. lbs.}$$

$$\frac{M_R}{t} = 50.18 \qquad \frac{M_L}{t} = 74.08$$

Points of Contraflexure:

$$Y_R = \frac{M_R}{H_R} = \frac{1931.80}{10.611} = 182.1 \text{ in.} \qquad Y_L = 12H_L \pm \sqrt{144H_L^2 - 24M_L} = 142.3 \text{ in.}$$

$$\frac{Y}{d} = \frac{182.1}{324} = 0.562 \qquad \frac{Y}{d} = \frac{142.3}{324} = 0.439$$

Span 40 ft. - Col. height 16 ft.

Total height of structure $t = 16 + 10 = 26$ ft.
Normal load on roof segment $5.5902 \times 0.7455 = 4.167$ lbs.

Class I.

Case A:
Total Horizontal load $H = 16x1 + 10x0.7455 = 23.455$ lbs.

$d_B = 32,609$ $\quad d_G \sin A = 14,208 \times 0.4472 = 6354$
See Fig. 68

J:	15,924 x 3 =	47,772		J:	7154 x 2.084 =	14,909
G:	14,208 x $5\frac{1}{2}$ =	78,144		E:	8624 x 4.167 =	35,936
F:	8,434 x 5 =	42,170		3:	9024 x 4.167 =	37,603
		168,086		6:	9204 x 4.167 =	38,353
				7:	8904 x 2.084 =	18,556
						145,357

$168,086 + 145,357 = 313,443$

$H_R = \frac{313,443}{32,609} = 9.612$ lbs. $\qquad H_L = H - H_R = 13.843$ lbs.

$\frac{H_R}{t} = 0.370 \qquad \frac{H_L}{t} = 0.532 \qquad \frac{H_R}{H} = 0.410$

Case B:
Total Vertical load $V = 20 \times 0.7455 = 14.910$ lbs.

$$V_R = \frac{16x8 + 0.7455x22.36x18.30}{40} = \frac{128 + 305.050}{40} = 10.826 \text{ lbs.}$$

$\frac{V_R}{t} = 0.416 \qquad V_L = V - V_R = 4.094$ lbs. $\qquad \frac{V_L}{t} = 0.157$

Class II

Case A:
$d_B = 9,445$ $\quad d_G \sin A = 3394 \times 0.4472 = 1518$
See Fig. 73

J:	4450 x 3	= 13,350
G:	3390 x $5\frac{1}{2}$	= 18,667
F:	1292 x 5	= 6,460
		38,477

J:	1998 x 2.084	= 4,164
2:	2958 x 4.167	= 12,326
3:	3218 x 4.167	= 13,409
6:	3358 x 4.167	= 13,993
7:	3138 x 2.084	= 6,540
		50,432

38,477 + 50,432 = 88,909

$H_R = \frac{88,909}{9,445} = 9.413$ lbs. $\qquad H_L = H - H_R = 14.042$ lbs.

$\frac{H_R}{t} = 0.362 \qquad \frac{H_L}{t} = 0.540 \qquad \frac{H_R}{H} = 0.401$

Case B:

$d_B = 211,776$ $\qquad$ See Fig. 77.

J:	51,925 x 3	= 155,800
G:	21,281 x $5\frac{1}{2}$	= 117,000
F:	5,320 x 5	= 26,600
		299,400

J:	23,000 x 2.084	= 47,900
2:	51,000 x 4.167	= 212,500
3:	80,000 x 4.167	= 333,600
6:	108,000 x 4.167	= 450,000
7:	137,000 x 2.084	= 285,500
		1,329,500

299,400 + 1,329,500 = 1,628,900

$V_R = \frac{1,628,900}{211,776} = 7.692$ lbs. $\qquad V_L = V - V_R = 7.218$ lbs.

$\frac{V_R}{t} = 0.296 \qquad \frac{V_L}{t} = 0.278$

Case C:

$d_K + \Delta_4 = 101.27 \qquad d_G = \sin A = 10.38$

See Fig. 83.

J:	29.81 x 3	= 89.43
G:	23.20 x $5\frac{1}{2}$	=127.60
F:	8.90 x 5	= 44450
		261.53

J:	13.38 x 2.084	= 27.88
2:	19.23 x 4.167	= 80.13
3:	20.58 x 4.167	= 85.76
6:	20.98 x 4.167	= 87.42
7:	19.38 x 2.084	= 40.39
		321.58

261.53 + 321.58 = 583.11

$$M_R = \frac{120x583.11}{101.27} = \frac{69,973.20}{101.27} = 690.96 \text{ in. lbs.}$$

$$M_L = \sum Pd - 480\ V_R - M_R$$

$$= (16x8x12) + (0.7455x22.36x18.30x12) - (480x7.692) - 690.96$$

$$= 1536 + 3660.60 - 3692.16 - 690.96 = 813.48 \text{ in. lbs.}$$

$\frac{M_R}{t} = 26.58$ $\frac{M_L}{t} = 31.29$

Points of Contraflexure:

$Y_R = \frac{M_R}{H_R} = \frac{690.96}{9.413} = 73.4$ in. $Y_L = 12H_L \pm \sqrt{144H_L^2 - 24M_L} = 74.5$ in.

$\frac{Y}{d} = \frac{73.4}{120} = 0.612$ $\frac{Y}{dd} = \frac{74.5}{120} = 0.621$

Span 40 ft. - Col. height 21 ft.

Total height of structure	t = 21 + 10 = 31 ft.
Normal load on roof segment	5.5902 x 0.7455 = 4.167 lbs.

Class I

Case A: (Fig. 67)

Total Horizontal load H = 21x1 + 10x0.7455 = 28.455 lbs.

d_B = 83,483 d_G sin A = 39,060 x 0.4472 = 17,470

See Fig. 69.

J:	41,276 x 3	=	123,830
G:	39,060 x 5½	=	214,830
F:	30,473 x 5	=	152,360
E:	16,567 x 5	=	82,830
			573,850

J:	18,500 x 2.084	=	38,550
2:	20,370 x 4.167	=	84,880
3:	20,900 x 4.167	=	87,090
6:	21,140 x 4.167	=	88,090
7:	20,670 x 2.084	=	43,080
			341,690

573 + 341,690 = 915,540

$H_R = \frac{915,540}{83,483} = 10.967$ lbs. $H_L = 17.488$ lbs.

$\frac{H_R}{t} = 0.354$ $\frac{H_L}{t} = 0.564$ $\frac{H_R}{H} = 0.385$

Case B:

Total Vertical load $V = 20 \times 0.7455 = 14.910$ lbs.

$$V_R = \frac{21 \times 10.5 + 0.7455 \times 22.36 \times 20.55}{40} = \frac{220.5 + 342.556}{40} = 14.076 \text{ lbs.}$$

$\frac{V_R}{t} = 0.454$ $V_L = V - V_R = 0.834$ lbs. $\frac{V_L}{t} = 0.0269$

Class II

Case A: (Fig. 72)

$d_B = 22,346$

J:	10,860	x 3	=	32,580
G:	9,545	x $5\frac{1}{2}$	=	52,500
F:	6,015	x 5	=	30,070
E:	1,947	x 5	=	9,740
				124,890

$d_G \sin A = 4270$

See Fig. 74

J:	4870	x 2.084	=	10,150
2:	6000	x 4.167	=	25,000
3:	6310	x 4.167	=	26,290
6:	6460	x 4.167	=	26,920
7:	6230	x 2.084	=	12,980
				101,340

$124,890 + 101,340 = 226,230$

$H_R = \frac{226,230}{22,346} = 10.124$ lbs. $H_L = H - H_R = 18.331$ lbs.

$\frac{H_R}{t} = 0.327$ $\frac{H_R}{t} = 0.591$ $\frac{H_R}{H} = 0.356$

Case B:

$d_B = 296,899$

J:	91,295	x 3	=	273,900
G:	47,882	x $5\frac{1}{2}$	=	263,400
F:	21,281	x 5	=	106,400
E:	5,320	x 5	=	26,600
				670,300

See Fig. 78 and 81.

J:	42,000	x 2.084	=	87,500
2:	81,000	x 4.167	=	337,500
3:	123,000	x 4.167	=	512,500
6:	165,000	x 4.167	=	687,600
7:	208,000	x 2.084	=	433,500
				2,058,600

$670,300 + 2,058.600 = 2,728,900$

$V_R = \frac{2,728,900}{296,899} = 9.191$ lbs. $V_L = V - V_R = 5.719$ lbs.

$\frac{V_R}{t} = 0.296$ $\frac{V_L}{t} = 0.184$

Case C: (Fig. 82)

$d_K + \Delta_4 = 198.99$

$d_G \sin A = 22.41$

See Fig. 84.

J:	56.39	x	3	=	169.17
G:	50.12	x	$5\frac{1}{2}$	=	275.66
F:	31.74	x	5	=	158.70
E:	10.30	x	5	=	51.50
					655.03

J:	25.26	x	2.084	=	52.64
2:	31.06	x	4.167	=	129.43
3:	31.96	x	4.167	=	133.18
6:	32.42	x	4.167	=	135.09
7:	31.01	x	2.084	=	64.62
					514.96

$$655.03 + 514.96 = 1169.99$$

$$M_R = \frac{180 \times 1169.99}{198.99} = \frac{210{,}598.20}{198.99} = 1058.34 \text{ in. lbs.}$$

$$M_L = \sum Pd - 480V_R - M_R$$

$$= (21 \times 10.5 \times 12) + (0.7455 \times 22.36 \times 20.55 \times 12) - (480 \times 9.191) - 1058.34$$

$$= 2646 + 4110.67 - 4411.68 - 1058.34 = 1286.65 \text{ in. lbs.}$$

$$\frac{M_R}{t} = 34.14 \qquad \frac{M_L}{t} = 41.51$$

Points of contraflexure:

$$Y_R = \frac{M_R}{H_R} = \frac{1058.34}{10.124} = 104.54 \text{ in.} \qquad Y_L = 12H_L \pm \sqrt{144H_L^2 - 24M_L} = 87.8 \text{ in.}$$

$$\frac{Y}{d} = \frac{104.54}{180} = 0.581 \qquad \frac{Y}{d} = \frac{87.8}{180} = 0.488$$

Span 40 ft. Col. height 26 ft.

Total height of structure $t = 26 + 10 = 36$ ft.

Normal load on roof segment $5.5902 \times 0.7455 = 4.167$ lbs.

Class I

Case A:

Total Horizontal load $H = 26 \times 1 + 10 \times 0.7455 = 33.455$ lbs.

$d_B = 172{,}255 \qquad d_G \sin A = 82{,}855 \times 0.4472 = 37{,}050$

See Fig. 70.

J:	85,579 x 3	=	256,740	J:	38,300 x 2.084	=	79,820
G:	82,855 x $5\frac{1}{2}$	=	455,700	2:	40,650 x 4.167	=	169,390
F:	71,451 x 5	=	357,260	3:	41,275 x 4.167	=	171,990
E:	52,068 x 5	=	260,340	6:	41,600 x 4.167	=	173,350
Q:	27,364 x 5	=	136,820	7:	41,050 x 2.084	=	85,550
			1,466,860				680,100

$$1,466,860 + 680,100 = 2,146,960$$

$$H_R = \frac{2,146,960}{172,255} = 12.464 \text{ lbs.} \qquad H_L = H - H_R = 20.991 \text{ lbs.}$$

$$\frac{H_R}{t} = 0.346 \qquad \frac{H_L}{t} = 0.583 \qquad \frac{H_R}{H} = 0.373$$

Case B:

Total Vertical load $\qquad V = 20x\ 0.7455 = 14.910$ lbs.

$$V_R = \frac{26x13+0.7455x22.36x22.80}{40} = \frac{338+380.062}{40} = 17.952 \text{ lbs.}$$

$$\frac{V_R}{t} = 0.499 \qquad V_L = V - V_R = -3.042 \text{ lbs.} \qquad \frac{V_L}{t} = -0.0845$$

Class II

Case A:

$d_B = 44,707 \qquad\qquad d_G \sin A = 9136$

See Fig. 75.

J:	21,997 x 3	=	65,991	J:	9866 x 2.084	=	20,561
G:	20,430 x $5\frac{1}{2}$	=	112,365	2:	11226 x 4.167	=	46,779
F:	15,482 x 5	=	77,410	3:	11611 x 4.167	=	48,383
E:	8,654 x 5	=	43,270	6:	11786 x 4.167	=	49,112
Q:	2,607 x 5	=	13,035	7:	11491 x 2.084	=	23,947
			312,071				188,782

$$312,071 + 188,782 = 500,853$$

$$H_R = \frac{500,853}{44,707} = 11.203 \text{ lbs.} \qquad H_L = H - H_R = 22.252 \text{ lbs.}$$

$$\frac{H_R}{t} = 0.311 \qquad \frac{H_L}{t} = 0.618 \qquad \frac{H_R}{H} = 0.335$$

Case B:

$d_B = 382,023 \qquad\qquad$ See Fig. 79.

J:	141,304	x 3	=	423,900	
G:	85,123	x $5\frac{1}{2}$	=	468,200	
F:	47,882	x 5	=	239,400	
E:	21,281	x 5	=	106,400	
Q:	5,320	x 5	=	26,600	
				1,264,500	

J:	63,000	x 2.084	=	131,300
2:	116,000	x 4.167	=	483,400
3:	169,000	x 4.167	=	704,200
6:	222,000	x 4.167	=	925,100
7:	275,000	x 2.084	=	573,100
				2,817,100

1,264,500 + 2,817,100 = 4,081,600

$V_R = \frac{4,081,600}{382,025} = 10.684$ lbs. $\quad V_L = V - V_R = 4.226$ lbs.

$\frac{V_R}{t} = 0.297 \qquad \frac{V_L}{t} = 0.117$

Case C:

$d_K + \Delta_4 = 332.11 \qquad d_G \sin A = 38.42$

See Fig. 85.

J:	91.86	x 3	=	275.58
G:	85.91	x $5\frac{1}{2}$	=	472.51
F:	65.32	x 5	=	326.60
E:	36.58	x 5	=	182.90
Q:	11.03	x 5	=	55.15
				1312.74

J:	41.17	x 2.084	=	85.80
2:	46.32	x 4.167	=	193.02
3:	47.42	x 4.167	=	197.60
6:	47.72	x 4.167	=	198.85
7:	46.30	x 2.084	=	96.49
				771.76

1312.74 + 771.76 = 2084.50

$M_R = \frac{240\text{x}2084.50}{332.11} = \frac{500,280.00}{332.11} = 1506.37$ in. lbs.

$M_L = \sum Pd - 480V_R - M_R$

$= (26\text{x}13\text{x}12) + (0.7455\text{x}22.36\text{x}22.80\text{x}12) - (480\text{x}10.684) - 1506.37$

$= 4056 + 4560.74 - 5128.32 - 1506.37 = 1982.05$ in. lbs.

$\frac{M_R}{t} = 41.84 \qquad \frac{M_L}{t} = 55.06$

Points of Contraflexure:

$Y_R = \frac{M_R}{H_R} = \frac{1506.37}{11.203} = 134.46$ in. $\qquad Y_L = 12H_L \pm \sqrt{144H_L^2 - 24M_L} = 113.0$ in.

$\frac{Y}{d} = \frac{134.46}{240} = 0.560 \qquad \frac{Y}{d} = \frac{113.0}{240} = 0.471$

Span 40 ft. - Col. height 31 ft.

Total height of structure $t = 31 + 10 = 41$ ft.
Normal load on roof segment $5.5902 \times 0.7455 = 4.167$ lbs.

Class I

Case A:

Total Horizontal load $H = 31\times1 + 10 \times 0.7455 = 38.455$ lbs.

$d_B = 135,181$

$d_G \sin A = 63,724\times0.4472 = 28.500$
See Fig. 71.

J:	66,953	x 3	=	200,860
G:	63,724	x $5\frac{1}{2}$	=	350,480
F:	56,817	x 5	=	284,080
E;	46,018	x 5	=	230,090
Q:	32,301	x 5	=	161,510
S:	16,657	x 5	=	83,190
				1,310,210

J:	30,000	x 2.084	=	62,520
2:	32,750	x 4.167	=	136,470
3:	33,450	x 4.167	=	139,390
6:	33,900	x 4.167	=	141,260
7:	33,230	x 2.084	=	69,250
				548,890

$1,310,210 + 548,890 = 1,859,100$

$H_R = \dfrac{1,859,100}{135,181} = 13.753$ lbs. $H_L = H - H_R = 24.702$ lbs.

$\dfrac{H_R}{t} = 0.335$ $\dfrac{H_L}{t} = 0.602$ $\dfrac{H_R}{H} = 0.358$

Case B:

Total Vertical load $V = 20 \times 0.7455 = 14.910$ lbs.

$V_R = \dfrac{31\times15.5 + 0.7455\times22.36\times25.05}{40} = \dfrac{480.5 + 417.568}{40} = 22.452$ lbs.

$\dfrac{V_R}{t} = 0.548$ $V_L = V - V_R = -7.542$ lbs. $\dfrac{V_L}{t} = -0.184$

Class II.

Case A:

$d_B = 35,685$

$d_G \sin A = 7033$
See Fig. 76

J;	17,458	x 3	=	53,374
G:	15,727	x $5\frac{1}{2}$	=	86,498
F:	12,660	x 5	=	63,300
E:	8,581	x 5	=	42,905
Q:	4,462	x 5	=	22,310
S:	1,278	x 5	=	6,390
				273,777

J:	7813	x 2.084	=	16,282
2:	9323	x 4.167	=	38,849
3:	9753	x 4.167	=	40,641
6:	9973	x 4.167	=	41,557
7:	9673	x 2.084	=	20,160
				157,489

273,777 + 157,489 = 431,266

$H_R = \frac{431,266}{35,685} = 12.085$ lbs. $H_L = H - H_R = 26.370$ lbs.

$\frac{H_R}{t} = 0.295$ $\frac{H_L}{t} = 0.643$ $\frac{H_R}{H} = 0.314$

Case B:

$d_B = 175,594$ See Fig. 80.

J:	73,868	x 3	=	221,600
G:	48,649	x $5\frac{1}{2}$	=	267,600
F:	31,135	x 5	=	155,600
E:	17,513	x 5	=	87,600
Q:	7,783	x 5	=	38,900
S:	1,946	x 5	=	9,700
				781,000

J:	32,500	x 2.084	=	67,700
2:	58,000	x 4.167	=	241,700
3:	82,500	x 4.167	=	343,800
6:	107,000	x 4.167	=	445,900
7:	131,500	x 2.084	=	274,000
				1,373,100

781,000 + 1,373,100 = 2,154,100

$V_R = \frac{2,154,100}{175,594} = 12.268$ lbs. $V_L = V - V_R = 2.642$ lbs.

$\frac{V_R}{t} = 0.299$ $\frac{V_L}{t} = 0.0644$

Case C:

$d_K + \Delta_4 = 196.29$ $d_G \sin A = 22.28$

See Fig. 86.

J:	54.76	x 3	=	164.28
G:	49.83	x $5\frac{1}{2}$	=	274.06
F:	40.33	x 5	=	201.65
E:	27.44	x 5	=	137.20
Q:	14.31	x 5	=	71.55
S:	4.10	x 5	=	20.50
				869.24

J:	24.53	x 2.084	=	51.12
2:	28.75	x 4.167	=	119.80
3:	29.68	x 4.167	=	123.68
6:	29.93	x 4.167	=	124.72
7:	28.58	x 2.084	=	59.56
				478.88

869.24 + 478.88 = 1,348.12

$$M_R = \frac{300 \times 1348.12}{196.29} = \frac{404,436.00}{196.29} = 2060.40 \text{ in. lbs.}$$

$M_L = \sum Pd - 480\ V_R - M_R$

$= (31 x 15.5 x 12) + (0.7455 x 22.36 x 25.05 x 12) - (480 x 12.268) - 2060.40$

$= 5766 + 5010.82 - 5888.64 - 2060.40 = 2827.78$ in. lbs.

$\frac{M_R}{t} = 49.77$ $\frac{M_L}{t} = 68.97$

Points of Contraflexure:

$Y_R = \frac{M_R}{H_R} = \frac{2060.40}{12.085} = 170.49$ in. $Y_L = 12H_L \pm \sqrt{144H_L^2 - 24M_L}$

$= 137.2$ in.

$\frac{Y}{d} = \frac{170.49}{300} = 0.568$ $\frac{Y}{d} = \frac{137.2}{300} = 0.457$

Span 50 ft. - Col. height 16 ft.

Total height of structure $t = 16 + 12.5 = 28.5$ ft.
Normal load on roof segment $6.9878 \times 0.7455 = 5.209$ lbs.

Class I.

Case A:
Total Horizontal load $H = 16\times1 + 12.5\times0.7455 = 25.319$ lbs.

$d_B = 29{,}654$ $\qquad$ $d_G \sin A = 13{,}300 \times 0.4472 = 5950$
See Fig. 88.

J:	14,470 x 3 = 43,410		J:	6510 x 2.605 = 16,960
G:	13,300 x $5\frac{1}{2}$ = 73,150		2:	7730 x 5.209 = 40,270
F:	7,980 x 5 = 39,900		3:	8080 x 5.209 = 42,090
	156,460		6:	8230 x 5.209 = 42,870
			7:	7900 x 2.605 = 20,580
				162,770

$$156{,}460 + 162{,}770 = 319{,}230$$

$$H_R = \frac{319{,}230}{29{,}654} = 10.765 \text{ lbs.} \qquad H_L = H - H_R = 14.554 \text{ lbs.}$$

$$\frac{H_R}{t} = 0.378 \qquad \frac{H_L}{t} = 0.511 - \qquad \frac{H_R}{H} = 0.425$$

Case B:
Total Vertical load $V = 25 \times 0.7455 = 18.638$ lbs.

$$V_R = \frac{16\times8 + 0.7455\times27.95\times21.20}{50} = \frac{128 + 441.738}{50} = 11.395 \text{ lbs.}$$

$$\frac{V_R}{t} = 0.400 \qquad V_L = V - V_R = 7.243 \text{ lbs.} \qquad \frac{V_L}{t} = 0.254$$

Class II

Case A:
$d_B = 8{,}369$ $\qquad$ $d_G \sin A = 3{,}175\times0.4472 = 1420$
See Fig. 93

$$\begin{aligned} J&: 3{,}916 \times 3 = 11{,}748 \\ G&: 3{,}175 \times 5\tfrac{1}{2} = 17{,}463 \\ F&: 1{,}237 \times 5 = \underline{6{,}185} \\ & \qquad 35{,}396 \end{aligned}$$

$$\begin{aligned} J&: 1760 \times 2.605 = 4{,}585 \\ 2&: 2580 \times 5.209 = 13{,}439 \\ 3&: 2845 \times 5.209 = 14{,}820 \\ 6&: 2930 \times 5.209 = 15{,}262 \\ 7&: 2805 \times 2.605 = \underline{7{,}307} \\ & \qquad 55{,}413 \end{aligned}$$

$$35{,}396 + 55{,}413 = 90{,}809$$

$$H_R = \frac{90{,}809}{8{,}369} = 10.851 \text{ lbs.} \qquad H_L = H - H_R = 14.468 \text{ lbs.}$$

$$\frac{H_R}{t} = 0.381 \qquad \frac{H_L}{t} = 0.508 \qquad \frac{H_R}{H} = 0.429$$

Case B:

$$d_B = 327{,}902$$

See Fig. 97.

$$\begin{aligned} J&: 64{,}906 \times 3 = 194{,}700 \\ G&: 26{,}601 \times 5\tfrac{1}{2} = 146{,}300 \\ F&: 6{,}650 \times 5 = \underline{33{,}200} \\ & \qquad 374{,}200 \end{aligned}$$

$$\begin{aligned} J&: 27{,}500 \times 2.605 = 71{,}700 \\ 2&: 72{,}000 \times 5.209 = 375{,}000 \\ 3&: 119{,}000 \times 5.209 = 619{,}900 \\ 6&: 163{,}000 \times 5.209 = 849{,}100 \\ 7&: 211{,}000 \times 2.605 = \underline{549{,}600} \\ & \qquad 2465{,}300 \end{aligned}$$

$$374{,}200 + 2{,}465{,}300 = 2{,}839{,}500$$

$$V_R = \frac{2{,}839{,}500}{327{,}902} = 8.660 \text{ lbs.} \qquad V_L = V - V_R = 9.978 \text{ lbs.}$$

$$\frac{V_R}{t} = 0.304 \qquad \frac{V_L}{t} = 0.350$$

Case C:

$$d_K + \Delta_4 = 97.47 \qquad d_G \sin A = 10.30$$

See Fig. 103.

$$\begin{aligned} J&: 28.03 \times 3 = 84.09 \\ G&: 23.04 \times 5\tfrac{1}{2} = 126.72 \\ F&: 9.02 \times 5 = \underline{45.10} \\ & \qquad 255.91 \end{aligned}$$

$$\begin{aligned} J&: 12.60 \times 2.605 = 32.82 \\ 2&: 18.00 \times 5.209 = 93.76 \\ 3&: 19.45 \times 5.209 = 101.32 \\ 6&: 19.90 \times 5.209 = 103.66 \\ 7&: 18.50 \times 2.605 = \underline{48.19} \\ & \qquad 379.75 \end{aligned}$$

$$255.91 + 379.75 = 635.66$$

$$M_R = \frac{120 \times 635.66}{97.47} = \frac{76{,}279.20}{97.47} = 782.59 \text{ in. lbs.}$$

$M_L = \sum Pd - 600V_R - M_R$

$= (16x8x12) + (0.7455x27.95x21.20x12) - (6\ 0x8.660) - 782.59$

$= 1536 + 5300.86 - 5196.00 - 782.59 = 858.27$ in. lbs.

$\frac{M_R}{t} = 27.46$ $\frac{M_L}{t} = 30.11$

Points of Contraflexure:

$Y_R = \frac{M_R}{H_R} = \frac{782.59}{10.851} = 72.12$ in. $Y_L = 12H_L \pm \sqrt{144H_L^2 - 24M_L} = 76.1$ in.

$\frac{Y}{d} = \frac{72.12}{120} = 0.601$ $\frac{Y}{d} = \frac{76.1}{120} = 0.634$

Span 50 ft. - Col. height 21 ft.

Total height of structure $t = 21 + 12.5 = 33.5$ ft.
Normal load on roof segment $6.9878 \times 0.7455 = 5.209$ lbs.

Class I

Case A: (Fig. 87)
Total Horizontal load $H = 21x1 + 12.5x0.7455 = 30.319$ lbs.

$d_B = 78,455$ $d_G \sin A = 37,291 \times 0.4472 = 16,680$
See Fig. 89.

J:	38,800	x	3	=	116,400
G:	37,291	x	$5\frac{1}{2}$	=	205,100
F:	29,294	x	5	=	146,470
E:	15,977	x	5	=	79,890
					547,860

J:	17,400	x	2.605	=	45,330
2:	18,950	x	5.209	=	98,710
3:	19,380	x	5.209	=	100,950
6:	19,600	x	5.209	=	102,100
7:	19,200	x	2.605	=	50,020
					397,110

$547,860 + 397,110 = 944,970$

$H_R = \dfrac{944,970}{78,455} = 12.045$ lbs. $\quad H_L = H - H_R = 18.274$ lbs.

$\dfrac{H_R}{t} = 0.360 \qquad \dfrac{H_L}{t} = 0.546 \qquad \dfrac{H_R}{H} \doteq 0.397$

Case B:

Total Vertical load $\quad V = 25 \times 0.7455 = 18.638$ lbs.

$$V_R = \frac{21 \times 10.5 + 0.7455 \times 27.95 \times 23.45}{50} = \frac{220.5 + 488.621}{50} = 14.182 \text{ lbs.}$$

$\dfrac{V_R}{t} = 0.423 \qquad V_L = V - V_R = 4.456$ lbs. $\qquad \dfrac{V_L}{t} = 0.135$

Class II

Case A: (Fig. 92)

$d_B = 20,639 \qquad d_G \sin A = 9103 \times 0.4472 = 4070$

See Fig. 94

J:	10,016	x 3	=	30,050
G:	9,103	x $5\frac{1}{2}$	=	50,070
F:	5,820	x 5	=	29,100
E:	1,900	x 5	=	9,500
				118,720

J:	4500	x 2.605	=	11,720
2:	5480	x 5.209	=	28,550
3:	5760	x 5.209	=	30,000
6:	5890	x 5.209	=	30,680
7:	5685	x 2.605	=	14,810
				115,760

$118,720 + 115,760 = 234,480$

$H_R = \dfrac{234,480}{20,639} = 11.361$ lbs. $\quad H_L = H - H_R = 18.958$ lbs.

$\dfrac{H_R}{t} = 0.339 \qquad \dfrac{H_L}{t} = 0.566 \qquad \dfrac{H_R}{H} = 0.375$

Case B:

$d_B = 460,907 \qquad$ See Fig. 98 and 101

J:	114,118	x 3	=	342,400
G:	59,852	x $5\frac{1}{2}$	=	329,200
F:	26,601	x 5	=	133,000
E:	6,650	x 5	=	33,200
				837,800

J:	52,000	x 2.605	=	135,500
2:	117,000	x 5.209	=	609,500
3:	180,000	x 5.209	=	937,600
6:	243,000	x 5.209	=	1265,800
7:	309,000	x 2.605	=	804,900
				3,753,300

$837,800 + 3,753,300 = 4,591,100$

$V_R = \frac{4,591,100}{460,907} = 9.961$ lbs. $\qquad V_L = V - V_R = 8.677$ lbs.

$\frac{V_R}{t} = 0.297 \qquad \frac{V_L}{t} = 0.259$

Case C: (Fig. 102)

$d_K + \Delta_4 = 193.77 \qquad d_G \sin A = 22.21$

See Fig. 104.

J:	54.24	x 3	=	162.72
G:	49.66	x $5\frac{1}{2}$	=	273.13
F:	31.84	x 5	=	159.20
E:	10.40	x 5	=	52.00
				647.05

J:	24.36	x 2.605	=	63.46
2:	29.11	x 5.209	=	151.63
3:	30.36	x 5.209	=	158.15
6:	30.81	x 5.209	=	160.48
7:	29.41	x 2.605	=	76.61
				610.33

647.05 + 610.33 = 1,257.38

$M_R = \frac{180 \times 1257.38}{193.77} = \frac{226,328.40}{193.77} = 1168.03$ in.lbs.

$M_L = \sum Pd - 600V_R - M_R$

$= (21 \times 10.5 \times 12) + (0.7455 \times 27.95 \times 23.45 \times 12) - (600 \times 9.61) - 1168.03$

$= 2646 + 5863.45 - 5976.60 - 1168.03 = 1364.82$ in. lbs.

$\frac{M_R}{t} = 34.87 \qquad \frac{M_L}{t} = 40.74$

Points of Contraflexure:

$Y_R = \frac{M_R}{H_R} = \frac{1168.03}{11.361} = 102.81$ in. $\qquad Y_L = 12H \pm \sqrt{144H_L^2 - 24\ M_L}$

$= 89.8$ in.

$\frac{Y}{d} = \frac{102.81}{180} = 0.57\bar{1} \qquad \frac{Y}{d} = \frac{89.8}{180} = 0.498$

Span 50 ft. - Col. height 26 ft.

Total height of structure $\quad t = 26 + 12.5 = 38.5$ ft.

Normal load on roof segment $\quad 6987 \times 0.7544 = 5.209$ lbs.

Class I

Case A:
Total horizontal load   H = 26x1+12.5 x 0.7455 = 35.319 lbs.

d_B = 164,564 d_G sin A = 79,936 x 0.4472 = 35,750
See Fig. 90.

J:	81,784 x 3 = 245,350		J:	36,610 x 2.605 = 95,370	
G:	79,936 x 5½ = 439,650		2:	38,500 x 5.209 = 200,550	
F:	69,262 x 5 = 346,310		3:	39,050 x 5.209 = 203,410	
E:	50,608 x 5 = 253,040		6:	39,290 x 5.209 = 204,660	
Q:	26,634 x 5 = 133,170		7:	38,780 x 2.605 = 101,020	
	1417,520			805,010	

1,417,520 + 805,010 = 2,222,530

$$H_R = \frac{2,222,530}{164,564} = 13.506 \text{ lbs.} \qquad H_L = H - H_R = 21.813 \text{ lbs.}$$

$$\frac{H_R}{t} = 0.351 \qquad \frac{H_L}{t} = 0.567 \qquad \frac{H_R}{H} = 0.382$$

Case B:
Total Vertical load   V = 25 x 0.7455 = 18.638 lbs.

$$V_R = \frac{26x13+0.7455x27.95x25.60}{50} = \frac{338 + 533.420}{50} = 17.428 \text{ lbs.}$$

$$\frac{V_R}{t} = 0.453 \qquad V_L = V - V_R = 1.210 \text{ lbs.} \qquad \frac{V_L}{t} = 0.0315$$

Class II

Case A:
d_B = 42,234 d_G sin A = 19,696 x 0.4472 = 8808
See Fig. 95

J:	20,778 x 3 = 62,330		J:	9,308 x 2.605 = 24,250	
G:	19,696 x 5½ = 108,330		2:	10,473 x 5.209 = 54,550	
F;	15,069 x 5 = 75,340		3:	10,808 x 5.209 = 56,300	
E:	8,471 x 5 = 42,360		6:	10,953 x 5.209 = 57,050	
Q:	2,561 x 5 = 12,800		7:	10,683 x 2.605 = 27,830	
	301,160			219,980	

301,160 + 219,980 = 521,140

$$H_R = \frac{521,140}{42,234} = 12.339 \text{ lbs.} \qquad H_L = H - H_R = 22.980 \text{ lbs.}$$

$$\frac{H_R}{t} = 0.320 \qquad \frac{H_L}{t} = 0.597 \qquad \frac{H_R}{H} = 0.349$$

Case B:

$d_B = 593,912$ See Fig. 99.

J: 176,631 x 3 = 529,900
G: 106,404 x $5\frac{1}{2}$ = 585,200
F: 59,852 x 5 = 299,300
E: 26,601 x 5 = 133,000
Q: 6,650 x 5 = 33,200
1,580,600

J: 78,000 x 2.605 = 203,200
2: 162,000 x 5.209 = 843,900
3: 245,000 x 5.209 = 1276,200
6: 330,000 x 5.209 = 1719,000
7: 415,000 x 2.605 = 1081,100
5,123,400

1,580,600 + 5,123,400 = 6,704,000

$$V_R = \frac{6,704,000}{593,912} = 11.288 \text{ lbs.} \qquad V_L = V - V_R = 7.350 \text{ lbs.}$$

$$\frac{V_R}{t} = 0.293 \qquad \frac{V_L}{t} = 0.191$$

Case C:

$d_K + \Delta_4 = 325.81$ $d_G \sin A = 38.07$
See Fig. 105

J: 89.42 x 3 = 268.26
G: 85.14 x $5\frac{1}{2}$ = 468.27
F: 65.28 x 5 = 326.40
E: 36.74 x 5 = 133.70
Q: 11.12 x 5 = 55.60
1,302.23

J: 40.02 x 2.605 = 104.25
2: 44.52 x 5.209 = 231.90
3: 45.67 x 5.209 = 237.90
6: 46.02 x 5.209 = 239.72
7: 44.67 x 2.605 = 116.37
930.14

1,302.23 + 930.14 = 2,232.37

$$M_R = \frac{240 \times 2,232.37}{325.8} = \frac{535,768.80}{325.81} = 1644.42 \text{ in. lbs.}$$

$$M_L = \sum Pd - 600V_R - M_R$$

$$= (26 \times 13 \times 12) + (0.7455 \times 27.95 \times 25.60 \times 12) - (600 \times 11.288) - 1644.42$$

$$= 4056 + 6401.04 - 6772.80 - 1644.42 = 2039.82 \text{ in. lbs.}$$

$$\frac{M_R}{t} = 42.71 \qquad \frac{M_L}{t} = 52.98$$

Points of Contraflexure:

$Y_R = \frac{M_R}{H_R} = \frac{1644.42}{12.339} = 133.27$ in. $\qquad Y_L = 12H_L \pm\sqrt{144H_L^2 - 24M_L} = 111.6$ in.

$\frac{Y}{d} = \frac{133.27}{240} = 0.555 \qquad \frac{Y}{d} = \frac{111.6}{240} = 0.465$

Span 50 ft. - Col. height 31 ft.

Total height of structure $t = 31 + 12.5 = 43.5$ ft.
Normal load on roof segment $6.9878 \times 0.7455 = 5.209$ lbs.

Class I

Case A:
Total Horizontal load $H = 31\text{x}1 + 12.5\text{x}0.7455 = 40.319$

$d_B = 124,285 \qquad d_G \sin A = 59,387\text{x}0.4472 = 26,560$
See Fig. 91.

J:	61,575 x 3 = 184,720	J:	27,560 x 2.605 = 71,790
G:	59,387 x $5\frac{1}{2}$ = 326,630	2:	29,840 x 5.209 = 155,440
F:	53,347 x 5 = 266,740	3:	30,460 x 5.209 = 158,670
E:	43,415 x 5 = 217,080	6:	30,730 x 5.209 = 160,070
Q:	30,565 x 5 = 152,820	7:	30,160 x 2.605 = 78,570
S:	15,769 x 5 = 78,840		
	1,226,830		624,540

1,226,830 + 624,540 = 1,851,370

$H_R = \frac{1,851,370}{124,285} = 14.896$ lbs. $\qquad H_L = H - H_R = 25.423$ lbs.

$\frac{H_R}{t} = 0.342 \qquad \frac{H_L}{t} = 0.584 \qquad \frac{H_R}{H} = 0.369$

Case B:
Total Vertical load $V = 25\text{x}0.7455 = 18.638$ lbs.

$V_R = \frac{31\text{x}15.5 + 0.7455\text{x}27.95\text{x}27.90}{50} = \frac{480.5 + 581.344}{50} = 21.237$ lbs.

$\frac{V_R}{t} = 0.488 \qquad V_L = V - V_R = -2.599$ lbs. $\qquad \frac{V_L}{t} = -0.0597$

Class II.

Case A:

$d_B = 32,589$ $d_G \sin A = 14,720 \times 0.4472 = 6580$

See Fig. 96.

J:	15,929 x 3	= 47,790		J:	7130 x 2.605	= 18,570
G:	14,720 x $5\frac{1}{2}$	= 80,960		2:	8440 x 5.209	= 43,960
F:	12,015 x 5	= 60,080		3:	8830 x 5.209	= 46,000
E:	8,218 x 5	= 41,090		6:	9000 x 5.209	= 46,880
Q:	4,301 x 5	= 21,500		7:	8720 x 2.605	= 22,720
S:	1,237 x 5	= 6,180				
		257,600				178,130

257,600 + 178,130 = 435,730

$H_R = \frac{435,730}{32,589} = 13.370$ lbs. $H_L = H - H_R = 26.949$ lbs.

$\frac{H_R}{t} = 0.307$ $\frac{H_L}{t} = 0.620$ $\frac{H_R}{H} = 0.332$

Case B:

$d_B = 271,377-$ See Fig. 100.

J:	92,335 x 3	= 277,000	J:	42,000 x 2.605	= 109,400
G:	60,811 x $5\frac{1}{2}$	= 334,500	2:	80,000 x 5.209	= 416,700
F:	38,919 x 5	= 194,600	3:	118,000 x 5.209	= 614,700
E:	21,892 x 5	= 109,500	6:	157,000 x 5.209	= 817,800
Q:	9,730 x 5	= 48,600	7:	196,000 x 2.605	= 510,600
S:	2,432 x 5	= 12,200			
		976,400			2,469,200

976,400 + 2,469,200 = 3,445,600

$V_R = \frac{3,445,600}{271,377} = 12.697$ lbs. $V_L = V - V_R = 5.941$ lbs.

$\frac{V_R}{t} = 0.292$ $\frac{V_L}{t} = 0.137$

Case C:

$d_K + \Delta_4 = 190.97$ $d_G \sin A = 21.82$

See Fig. 106.

J:	52.44 x 3	= 157.32	J:	23.52 x 2.605	= 61.27
G:	48.79 x $5\frac{1}{2}$	= 268.34	2:	27.32 x 5.209	= 142.31
F:	39.97 x 5	= 199.85	3:	28.27 x 5.209	= 147.26
E:	27.40 x 5	= 137.00	6:	28.52 x 5.209	= 148.56
Q:	14.37 x 5	= 71.85	7:	27.22 x 2.605	= 70.91
S:	4.14 x 5	= 20.70			
		855.06			570.31

$$855.06 + 570.31 = 1,425.37$$

$$M_R = \frac{300x1425.37}{190.97} = \frac{427,611.00}{190.97} = 2239.15 \text{ in. lbs.}$$

$$M_L = \sum Pd - 600V_R - M_R$$

$$= (31x15.5x12) + (0.7455x27.95x27.90x12) - (600x12.697) - 2239.15$$

$$= 5766 + 6976.13 - 7618.20 - 2239.15 = 2884.78 \text{ in. lbs.}$$

$$\frac{M_R}{t} = 51.47 \qquad \frac{M_L}{t} = 66.32$$

Points of Contraflexure:

$$Y_R = \frac{M_R}{H_R} = \frac{2239.15}{13.370} = 167.48 \text{ in.} \qquad Y_L = 12H_L \pm \sqrt{144H_L^2 - 24M} = 135.8 \text{in.}$$

$$\frac{Y}{d} = \frac{167.48}{300} = 0.558 \qquad \frac{Y}{d} = \frac{135.8}{300} = 0.452$$

Span 60 ft. - Col. height 16 ft.

Total height of structure $t = 16 + 15 = 31$ ft.
Normal load on roof segment $8.385 \times 0.7455 = 6.251$ lbs.

Class I

Case A:

Total Horizontal load $H = 16x1 + 15x0.7455 = 27,182$ lbs.

$d_B = 28,327$ $\qquad d_G \sin A = 12,895x0.4472 = 5770$

See Fig. 108.

J:	13,822 x 3 = 41,470		J:	6210 x 3.126 = 19,410
			2:	7490 x 6.251 = 46,820
			3:	7660 x 6.251 = 47,880
G:	12,895 x $5\frac{1}{2}$ = 70,920		6:	7820 x 6.251 = 48,880
F:	7,778 x 5 = 38,890		7:	7570 x 3.126 = 23,660
	151,280			186,650

$$151,280 + 186,650 = 337,930$$

$H_R = \dfrac{337,930}{28,327} = 11.930$ lbs. $\qquad H_L = H - H_R = 15.252$ lbs.

$\dfrac{H_R}{t} = 0.585 \qquad \dfrac{H_L}{t} = 0.492 \qquad \dfrac{H_R}{H} = 0.439$

Case B:

Total Vertical load $V = 30x0.7455 = 22.365$ lbs.

$V_R = \dfrac{16x8 + 0.7455x33.54x23.90}{60} = \dfrac{128 + 597.598}{60} = 12.093$ lbs.

$\dfrac{V_R}{t} = 0.390 \qquad V_L = V - V_R = 10.272$ lbs. $\qquad \dfrac{V_L}{t} = 0.331$

Class II.

Case A:

$d_B = 7,868$ $\qquad d_G \sin A = 3073 \times 0.4472 = 1374$

See Fig. 113.

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$$\begin{aligned}
J&: \; 3668 \times 3 = 11{,}000 \\
G&: \; 3075 \times 5\tfrac{1}{2} = 16{,}900 \\
F&: \; 1212 \times 5 = \underline{\;6{,}060\;} \\
&\qquad\qquad\qquad 33{,}960
\end{aligned}
\qquad
\begin{aligned}
J&: \; 1649 \times 3.126 = 5{,}150 \\
2&: \; 2424 \times 6.251 = 15{,}150 \\
3&: \; 2674 \times 6.251 = 16{,}720 \\
6&: \; 2799 \times 6.251 = 17{,}500 \\
7&: \; 2664 \times 3.126 = \underline{\;8{,}330\;} \\
&\qquad\qquad\qquad 62{,}850
\end{aligned}$$

$$33{,}960 + 62{,}850 = 96{,}810$$

$$H_R = \frac{96{,}810}{7{,}868} = 12.304 \text{ lbs.} \qquad H_L = H - H_R = 14.878 \text{ lbs.}$$

$$\frac{H_R}{t} = 0.397 \qquad \frac{H_L}{t} = 0.480 \qquad \frac{H_R}{H} = 0.453$$

Case B:

$d_B = 469{,}954$

See Fig.117

$$\begin{aligned}
J&: \; 77{,}888 \times 3 = 233{,}700 \\
G&: \; 31{,}921 \times 5\tfrac{1}{2} = 175{,}600 \\
F&: \; 7{,}980 \times 5 = \underline{\;39{,}900\;} \\
&\qquad\qquad\qquad 449{,}200
\end{aligned}
\qquad
\begin{aligned}
J&: \; 32{,}000 \times 3.126 = 100{,}000 \\
2&: \; 97{,}000 \times 6.251 = 606{,}300 \\
3&: \; 163{,}000 \times 6.251 = 1018{,}900 \\
6&: \; 229{,}000 \times 6.251 = 1431{,}500 \\
7&: \; 294{,}000 \times 3.126 = \underline{\;919{,}000\;} \\
&\qquad\qquad\qquad 4{,}075{,}700
\end{aligned}$$

$$449{,}200 + 4{,}075{,}700 = 4{,}524{,}900$$

$$V_R = \frac{4{,}524{,}900}{469{,}954} = 9.628 \text{ lbs.} \qquad V_L = V - V_R = 12.737 \text{ lbs.}$$

$$\frac{V_R}{t} = 0.311 \qquad \frac{V_L}{t} = 0.411$$

Case C:

$d_K + \Delta_4 = 95.50$ $\qquad$ $d_G \sin A = 10.27$

See Fig. 123

$$\begin{aligned}
J&: \; 27.12 \times 3 = 81.36 \\
G&: \; 22.97 \times 5\tfrac{1}{2} = 126.33 \\
F&: \; 9.09 \times 5 = \underline{\;45.45\;} \\
&\qquad\qquad\qquad 253.14
\end{aligned}
\qquad
\begin{aligned}
J&: \; 12.14 \times 3.126 = 37.95 \\
2&: \; 17.37 \times 6.251 = 108.58 \\
3&: \; 18.95 \times 6.251 = 118.46 \\
6&: \; 19.50 \times 6.251 = 121.89 \\
7&: \; 18.27 \times 3.126 = \underline{\;57.11\;} \\
&\qquad\qquad\qquad 443.99
\end{aligned}$$

$$253.14 + 443.99 = 697.13$$

$$M_R = \frac{120 \times 697.13}{95.50} = \frac{83{,}655.60}{95.50} = 875.97 \text{ in. lbs.}$$

$M_L = \sum Pd - 720V_R - M_R$

$= (16x8x12) + (0.7455x33.54x23.90x12) - (720x9.628) - 875.97$

$= 1536 + 7171.18 - 6932.16 - 875.97 = 899.05$

$\frac{M_R}{t} = 28.26$ $\qquad\qquad$ $\frac{M_L}{t} = 29.00$

Points of contraflexure:

$Y_R = \frac{M_R}{H_R} = \frac{875.97}{12.304} = 71.19$ in. $\qquad$ $Y_L = 12H_L \pm \sqrt{144H_L^2 - 24M_L} = 77.1$ in.

$\frac{Y}{d} = \frac{71.19}{120} = 0.593$ $\qquad\qquad$ $\frac{Y}{d} = \frac{77.1}{120} = 0.643$

Span 60 ft. - Col. height 21 ft.

Total height of structure $\qquad$ t = 21 + 15 = 36 ft.
Normal load on roof segment $\qquad$ 8.385 x 0.7455 = 6.251 lbs.

Class I

Case A: (Fig 107)
Total Horizontal load $\qquad$ H = 21x1+15x0.7455 = 32.182 lbs.

$d_B = 76{,}182$ $\qquad$ $d_G = \sin A = 36{,}497 \times 0.4472 = 16{,}320$
See Fig. 109

				J:	16,870	x 3.126	= 52,730
J:	37,689	x 3	= 113,070	2:	18,310	x 6.251	= 114,450
G:	36,497	x 5½	= 200,730	3:	18,720	x 6.251	= 117,020
F:	28,765	x 5	= 143,820	6:	18,930	x 6.251	= 118,330
E:	15,712	x 5	= 78,560	7:	18,560	x 3.126	= 58,020
			536,180				460,550

536,180 + 460,550 = 996,730

$H_R = \frac{996{,}730}{76{,}182} = 13.084$ lbs. $\qquad$ $H_L = H - H_R = 19.098$ lbs.

$\frac{H_R}{t} = 0.363$ $\qquad$ $\frac{H_L}{t} = 0.530$ $\qquad$ $\frac{H_R}{H} = 0.407$

Case B:

Total Vertical load $V = 30 \times 0.7455 = 22.365$ lbs.

$$V_R = \frac{21 \times 10.5 + 0.7455 \times 33.54 \times 26.15}{60} = \frac{220.5 + 653.857}{60} = 14.573 \text{ lbs.}$$

$\frac{V_R}{t} = 0.405$ $V_L = V - V_R = 7.792$ lbs. $\frac{V_L}{t} = 0.216$

Class II

Case A: (Fig. 112)

$d_B = 19{,}852$ $d_G \sin A = 8901 \times 0.4472 = 3980$

See Fig. 114

J:	9630	x 3	=	28,890
G:	8901	x 5½	=	48,960
F:	5729	x 5	=	28,640
E:	1876	x 5	=	9,380
				115,870

J:	4320	x 3.126	=	13,500
2:	5210	x 6.251	=	32,570
3:	5480	x 6.251	=	34,260
6:	5580	x 6.251	=	34,880
7:	5405	x 3.126	=	16,900
				132,110

$115{,}870 + 132{,}110 = 247{,}980$

$H_R = \frac{247{,}980}{19{,}852} = 12.491$ lbs. $H_L = H - H_R = 19.691$ lbs.

$\frac{H_R}{t} = 0.347$ $\frac{H_L}{t} = 0.547$ $\frac{H_R}{H} = 0.388$

Case B:

$d_B = 661{,}481$ correct See Fig. 118 and 121.

J:	136,492	x 3	=	409,500
G:	71,823	x 5½	=	395,000
F:	31,921	x 5	=	159,600
E:	7,980	x 5	=	39,900
				1,004,000

J:	60,000	x 3.126	=	187,600
2:	153,000	x 6.251	=	956,400
3:	245,000	x 6.251	=	1531,500
6:	338,000	x 6.251	=	2112,800
7:	435,000	x 3.126	=	1359,800
				6,148,100

$1{,}004{,}000 + 6{,}148{,}100 = 7{,}152{,}100$

$V_R = \frac{7{,}152{,}100}{661{,}481} = 10.812$ lbs. $V_L = V - V_R = 11.553$ lbs.

$\frac{V_R}{t} = 0.300$ $\frac{V_L}{t} = 0.321$

Case C: (Fig. 122)

$d_K + \triangle_4 = 191.18$ | $d_G \sin A = 22.23$
See Fig. 124.

J:	53.49 x 3 = 160.47		J:	23.98 x 3.126 = 74.96
G:	49.72 x $5\frac{1}{2}$ = 273.46		2:	28.63 x 6.251 = 178.97
F:	32.07 x 5 = 160.35		3:	29.98 x 6.251 = 187.40
E:	10.51 x 5 = 52.55		6:	30.43 x 6.251 = 190.22
	646.83		7:	29.38 x 3.126 = 91.84
				723.39

$$646.83 + 723.39 = 1,370.22$$

$$M_R = \frac{180 x 1370.22}{191.18} = \frac{246,639.60}{191.18} = 1290.09 \text{ in. lbs.}$$

$$M_L = \sum Pd - 720V_R - M_R$$

$$=(21x10.5x12)+(0.7455x33.54x26.15x12)-(720x10.812)-1290.09$$

$$= 2646 + 7846.28 - 7784.64 - 1290.09 = 1417.55 \text{ in. lbs.}$$

$$\frac{M_R}{t} = 35.84 \qquad \frac{M_L}{t} = 39.38$$

Points of Contraflexure:

$$Y_R = \frac{M_R}{H_R} = \frac{1290.09}{12.491} = 103.28 \text{ in.} \qquad Y_L = 12\ H_L \pm \sqrt{144H_L^2 - 24M_L} = 89.0 \text{ in.}$$

$$\frac{Y}{d} = \frac{103.28}{180} = 0.574 \qquad \frac{Y}{d} = \frac{89.0}{180} = 0.494$$

Span 60 ft. - Col. height 26 ft.

Total height of structure $t = 26 + 15 = 41$ ft.
Normal load on roof segment $8.385 \times 0.7455 = 6.251$ lbs.

Class I

Case A:
Total Horizontal load $H = 26x1+15x0.7455 = 37.182$ lbs.

$d_B = 161,137$ | $d_G \sin A = 78,646 \times 0.4472 = 35,170$
See Fig. 110.

J: 80,107 x 3 = 240,320		J: 35,850 x 3.126 = 112,070	
G: 78,646 x $5\frac{1}{2}$ = 432,550		2: 37,570 x 6.251 = 234,850	
F: 68,295 x 5 = 341,470		3: 38,090 x 6.251 = 238,100	
E: 49,963 x 5 = 249,810		6: 38,350 x 6.251 = 239,730	
Q: 26,312 x 5 = 131,560		7: 37,940 x 3.126 = 118,600	
1,395,710		943,350	

1,395,710 + 943,350 = 2,339,060

$H_R = \dfrac{2,339,060}{161,137} = 14.516$ lbs. $\quad H_L = H - H_R = 22.666$ lbs.

$\dfrac{H_R}{t} = 0.354 \qquad \dfrac{H_L}{t} = 0.553 \qquad \dfrac{H_R}{H} = 0.390$

Case B:

Total Vertical load $V = 30 \times 0.7455 = 22.365$ lbs.

$V_R = \dfrac{26 \times 13 + 0.7455 \times 33.54 \times 28.40}{60} = \dfrac{338 + 710.116}{60} = 17.469$ lbs.

$\dfrac{V_R}{t} = 0.426 \qquad V_L = V - V_R = 4.896$ lbs. $\qquad \dfrac{V_L}{t} = 0.119$

Class II

Case A:

$d_B = 41,103 \qquad d_G \sin A = 19,363 \times 0.4472 = 8660$

See Fig. 115

J: 20,225 x 3 = 60,670	J: 9,060 x 3.126 = 28,320
G: 19,363 x $5\frac{1}{2}$ = 106,500	2: 10,120 x 6.251 = 63,260
F: 14,882 x 5 = 74,410	3: 10,450 x 6.251 = 65,320
E: 8,387 x 5 = 41,930	6: 10,580 x 6.251 = 66,140
Q: 2,540 x 5 = 12,700	7: 10,345 x 3.126 = 32,340
296,210	255,380

296,210 + 255,380 = 551,590

$H_R = \dfrac{551,590}{41,103} = 13.420$ lbs. $\qquad H_L = H - H_R = 23.762$

$\dfrac{H_R}{t} = 0.327 \qquad \dfrac{H_L}{t} = 0.580 \qquad \dfrac{H_R}{H} = 0.361$

Case B:

$d_B = 853,009$ See Fig. 119.

J:	211,957 x 3 = 635,900		J:	95,000 x 3.126 = 297,000
G:	127,685 x $5\frac{1}{2}$ = 702,300		2:	215,000 x 6.251 = 1,344,000
F:	71,823 x 5 = 359,100		3:	335,000 x 6.251 = 2,094,100
E:	31,921 x 5 = 159,600		6:	460,000 x 6.251 = 2,875,500
Q:	7,980 x 5 = 39,900		7:	582,000 x 3.126 = 1,819,300
	1,896,800			8,429,900

$$1,896,800 + 8,429,900 = 10,326,700$$

$$V_R = \frac{10,326,700}{853,009} = 12.106 \text{ lbs.} \qquad V_L = V - V_R = 10.259 \text{ lbs.}$$

$$\frac{V_R}{t} = 0.295 \qquad \frac{V_L}{t} = 0.250$$

Case C:

$$d_K + \triangle_4 = 322.78 \qquad d_G \sin A = 37.90$$

See Fig. 125.

J:	88.22 x 3 = 264.66		J:	39.52 x 3.126 = 123.54
G:	84.75 x $5\frac{1}{2}$ = 466.12		2:	43.69 x 6.251 = 273.11
F:	65.23 x 5 = 326.15		3:	44.75 x 6.251 = 279.73
E:	36.80 x 5 = 184.00		6:	45.05 x 6.251 = 281.61
Q:	11.15 x 5 = 55.75		7:	43.93 x 3.126 = 137.33
	1296.68			1095.32

$$1296.68 + 1095.32 = 2392.00$$

$$M_R = \frac{240 \times 2392.00}{322.78} = \frac{574,080,00}{322.78} = 1778.55 \text{ in. lbs.}$$

$$M_L = \sum Pd - 720V_R - M_R$$

$$= (26 \times 13 \times 12) + (0.7455 \times 33.54 \times 28.40 \times 12) - (720 \times 12.106) - 1778.55$$

$$= 4056 + 8521.39 - 8716.32 - 1778.55 = 2082.52 \text{ in. lbs.}$$

$$\frac{M_R}{t} = 43.38 \qquad \frac{M_L}{t} = 50.79$$

Points of Contraflexure:

$$Y_R = \frac{M_R}{H_R} = \frac{1778.55}{13.420} = 132.53 \text{ in.} \qquad Y_L = 12H \pm \sqrt{144H_L^2 - 24M_L} = 109.0 \text{ in.}$$

$$\frac{Y}{d} = \frac{132.53}{240} = 0.558 \qquad \frac{Y}{d} = \frac{109.0}{240} = 0.454$$

Span 60 ft. - Col. height 31 ft.

Total height of structure $t = 31 + 15 = 46$ ft.
Normal load on roof segment $8.385 \times 0.7455 = 6.251$ lbs.

Class I

Case A:
Total Horizontal load $H = 31 \times 1 + 15 \times 0.7455 = 42.182$ lbs.

$d_B = 119,448$ $d_G \sin A = 57,474 \times 0.4472 = 25,700$
See Fig. 111

J:	59,203 x 3 = 177,610		J		
G:	57,474 x $5\frac{1}{2}$ = 316,110		J:	26,500 x 3.126 =	82,840
F:	51,817 x 5 = 259,080		2:	28,550 x 6.251 =	178,470
E:	42,268 x 5 = 211,340		3:	29,140 x 6.251 =	182,150
Q:	29,674 x 5 = 148,370		6:	29,420 x 6.251 =	183,900
S:	15,387 x 5 = 76,930		7:	28,980 x 3.126 =	90,590
	1189,440				717,950

$1189,440 + 717,950 = 1,907,390$

$H_R = \frac{1,907,390}{119,448} = 15.968$ lbs. $H_L = H - H_R = 26.214$ lbs.

$\frac{H_R}{t} = 0.347$ $\frac{H_L}{t} = 0.570$ $\frac{H_R}{H} = 0.379$

Case B:
Total Vertical load $V = 30 \times 0.7455 = 22.365$ lbs.

$V_R = \frac{31 \times 15.5 + 0.7455 \times 33.54 \times 30.65}{60} = \frac{480.5 + 766.376}{60} = 20.781$ lbs.

$\frac{V_R}{t} = 0.452$ $V_L = V - V_R = 1.584$ lbs. $\frac{V_L}{t} = 0.0344$

Class II.

Case A:
$d_B = 31,150$ $d_G \sin A = 14,256 \times 0.4472 = 6375$
See Fig. 116.

J:	15,224 x 3 = 45,670		J:	6815 x 3.126 = 21,300	
G:	14,256 x $5\frac{1}{2}$ = 78,410		2:	8015 x 6.251 = 50,100	
F:	11,718 x 5 = 58,590		3:	8355 x 6.251 = 52,230	
E:	8,051 x 5 = 40,250		6:	8515 x 6.251 = 53,230	
Q:	4,227 x 5 = 21,130		7:	8235 x 3.126 = 25,740	
S:	1,219 x 5 = 6,090				
	250,140			202,600	

250,140 + 202,600 = 452,740

$H_R = \frac{452,740}{31,150} = 14.534$ lbs. $H_L = H - H_R = 27.648$ lbs.

$\frac{H_R}{t} = 0.316$ $\frac{H_L}{t} = 0.601$ $\frac{H_R}{H} = 0.345$

Case B:

$d_B = 388,565$ See Fig. 120.

J:	110,802 x 3 = 332,400	J:	49,000 x 3.126 = 153,200
G:	72,973 x $5\frac{1}{2}$ = 401,400	2:	107,000 x 6.251 = 668,900
F:	46,703 x 5 = 233,500	3:	162,000 x 6.251 = 1012,700
E:	26,270 x 5 = 131,300	6:	218,000 x 6.251 = 1362,700
Q:	11,676 x 5 = 58,400	7:	273,000 x 3.126 = 853,400
S:	2,919 x 5 = 14,600		
	1171,600		4050,900

1171,600 + 4050,900 = 5,222,500

$V_R = \frac{5,222,500}{388,565} = 13.440$ lbs. $V_L = V - V_R = 8.925$ lbs.

$\frac{V_R}{t} = 0.292$ $\frac{V_L}{t} = 0.194$

Case C:

$d_K + \Delta_4 = 188.27$ $d_G \sin A = 21.60$

See Fig. 126.

J:	51.31 x 3 = 153.93	J:	23.00 x 3.126 = 71.90
G:	48.30 x $5\frac{1}{2}$ = 265.65	2:	26.55 x 6.251 = 165.96
F:	39.82 x 5 = 199.10	3:	27.45 x 6.251 = 171.59
E:	27.40 x 5 = 137.00	6:	27.80 x 6.251 = 173.78
Q:	14.40 x 5 = 72.00	7:	26.77 x 3.126 = 83.68
S:	4.16 x 5 = 20.80		
	848.48		666.91

848.48 + 666.91 = 1515.39

$$M_R = \frac{300 \times 1515.39}{188.27} = \frac{454,617.00}{188.27} = 2414.71 \text{ in. lbs.}$$

$$M_L = \sum Pd - 720V_R - M_R$$

$$= (31 \times 15.5 \times 12) + (0.7455 \times 33.54 \times 30.65 \times 12) - (720 \times 13.440) - 2414.71$$

$$= 5766 + 9196.51 - 9676.80 - 2414.71 = 2871.00 \text{ in. lbs.}$$

$$\frac{M_R}{t} = 52.49 \qquad \frac{M_L}{t} = 62.41$$

Points of Contraflexure:

$$Y_R = \frac{M_R}{H_R} = \frac{2414.71}{14.534} = 166.14 \text{ in.} \qquad Y_L = 12H_L \pm \sqrt{144H_L^2 - 24\ M_L}$$
$$= 128.8 \text{ in.}$$

$$\frac{Y}{d} = \frac{166.14}{300} = 0.554 \qquad \frac{Y}{d} = \frac{128.8}{300} = 0.429$$

Fig 1.

Fig. 2.

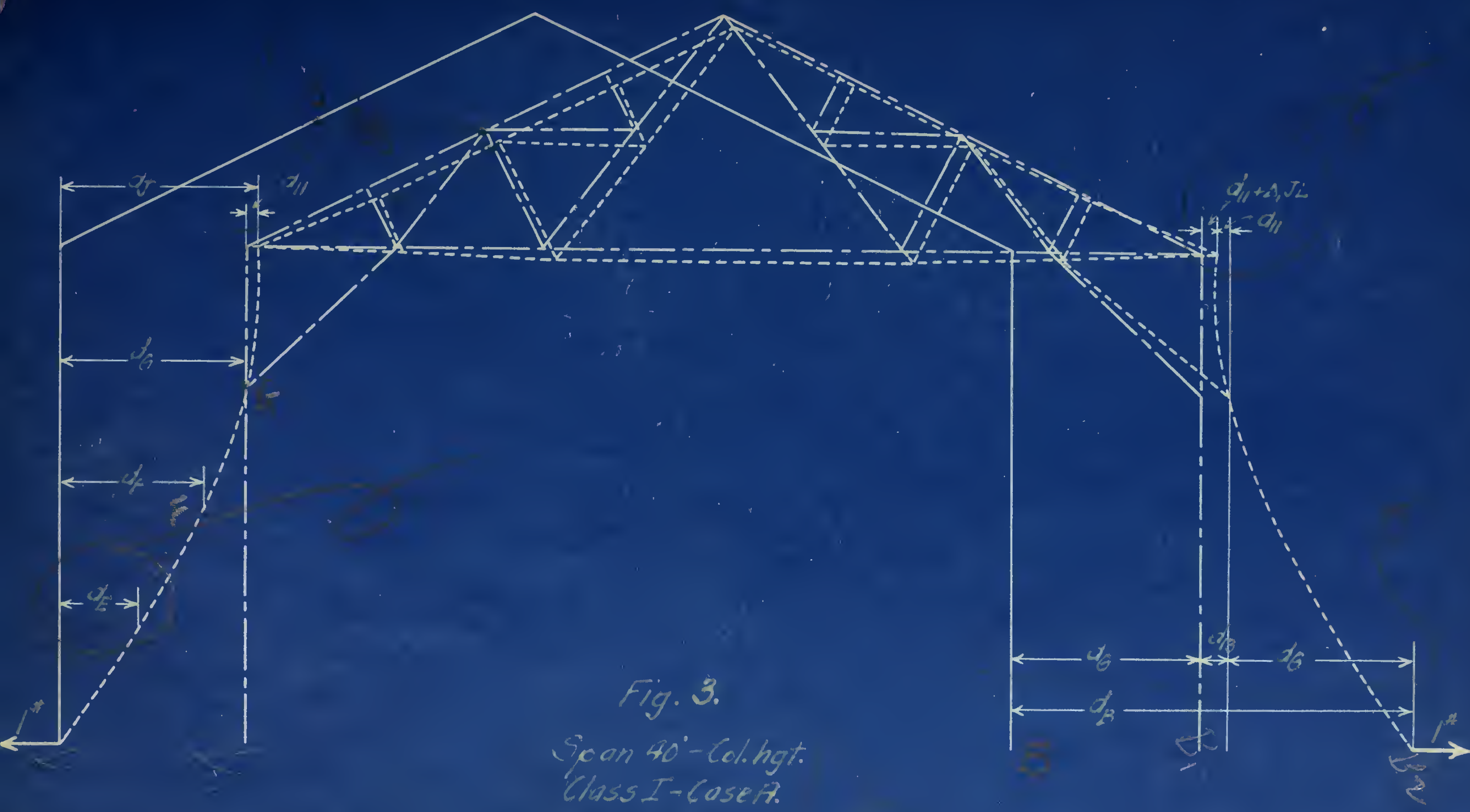
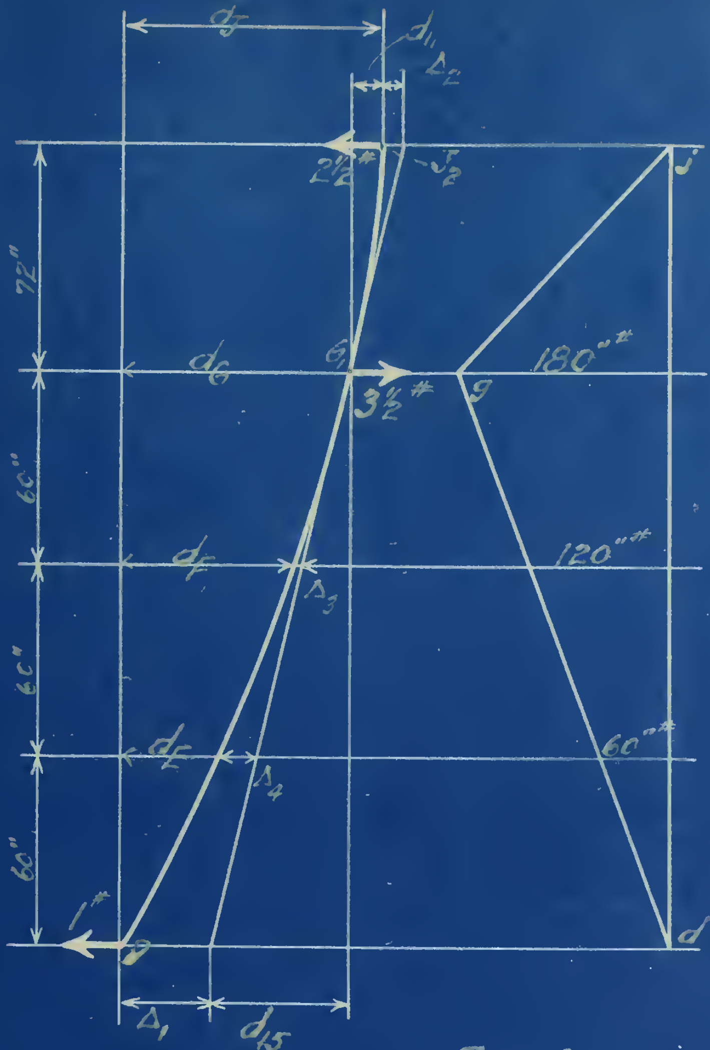


Fig. 3.
Span 40'-Col. hgt.
Class I-Case A.



d_7
d_{11}
Δ_2
$2\frac{1}{2}$#
J_2
72"
d_6
6_1
$3\frac{1}{2}$#
g
180"#
j
60"
d_F
Δ_3
120"#
60"
d_E
Δ_4
60"#
60"
1#
d
Δ_1
d_{15}

Fig. 4.

1#
1#
U_1
0.15"
0.15#

Fig. 5.

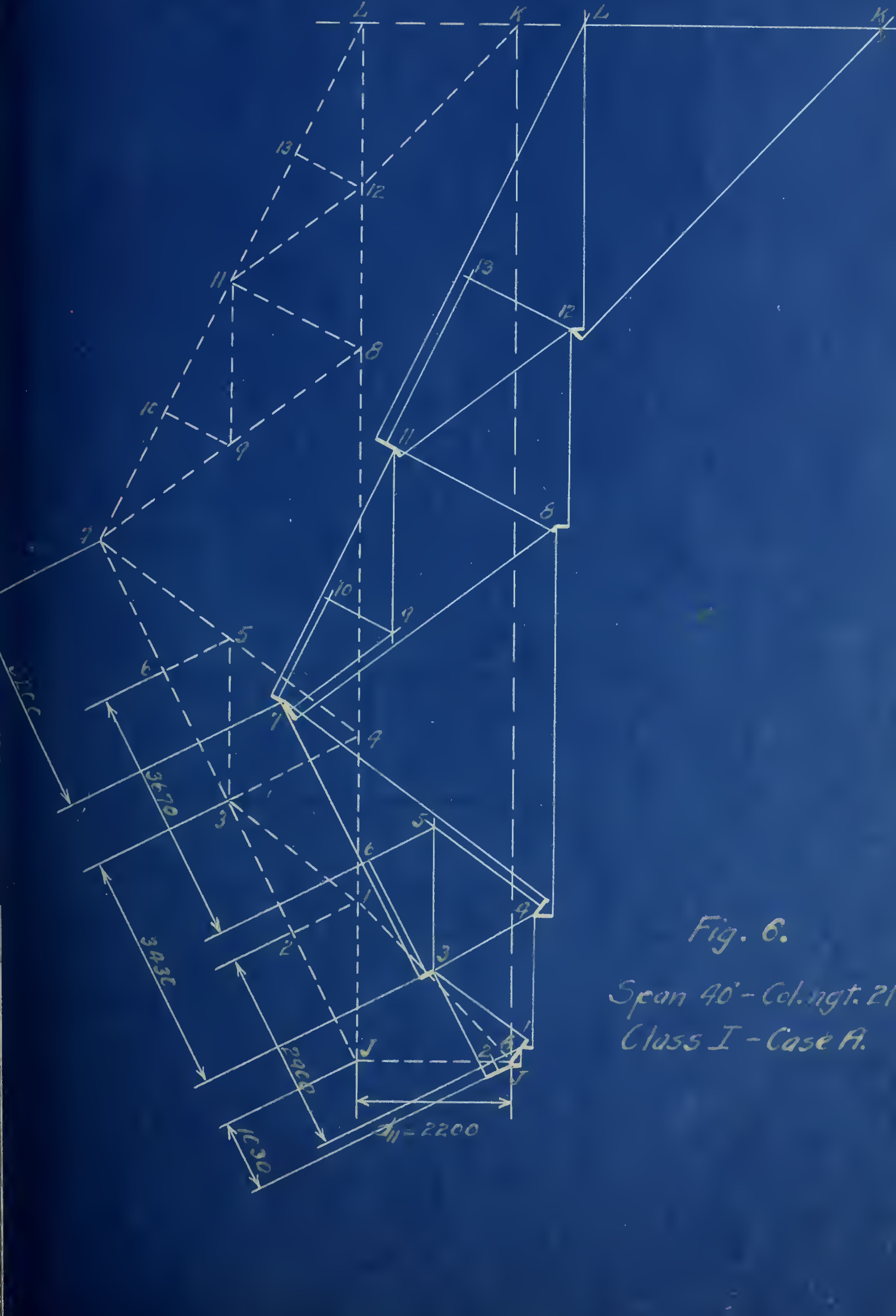


Fig. 6.

Span 40' - Col. hgt. 21'.
Class I - Case A.

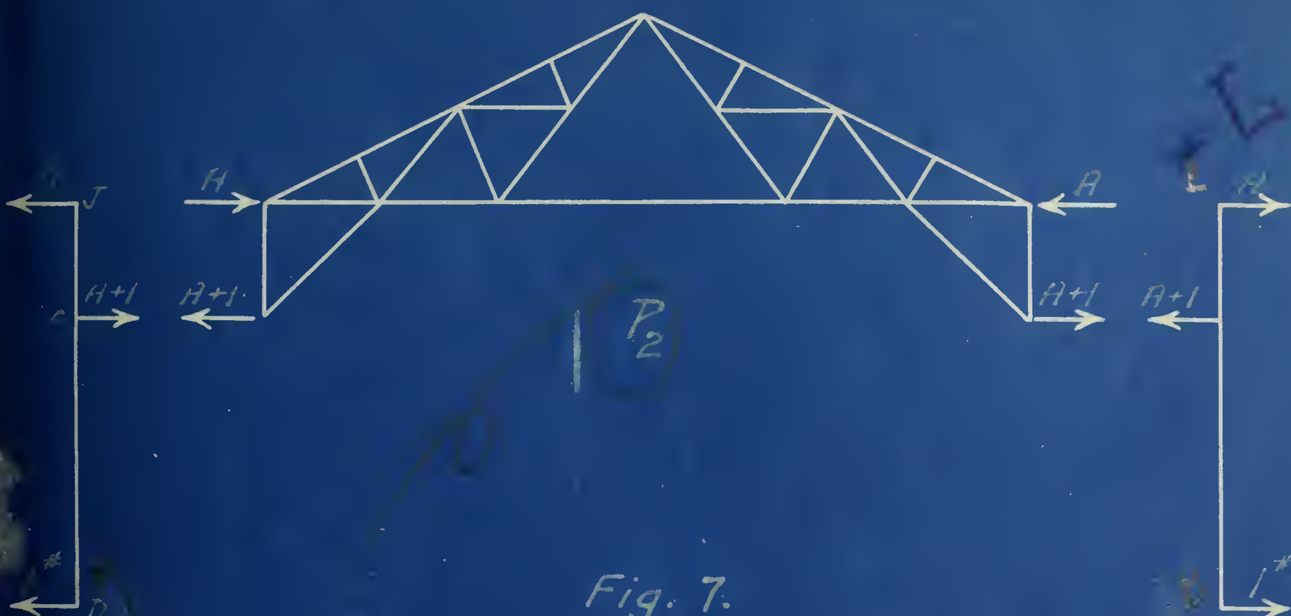

H
J
H+1
H+1
H+1
H+1
H
H
1*
D
1*
P2

Fig. 7.

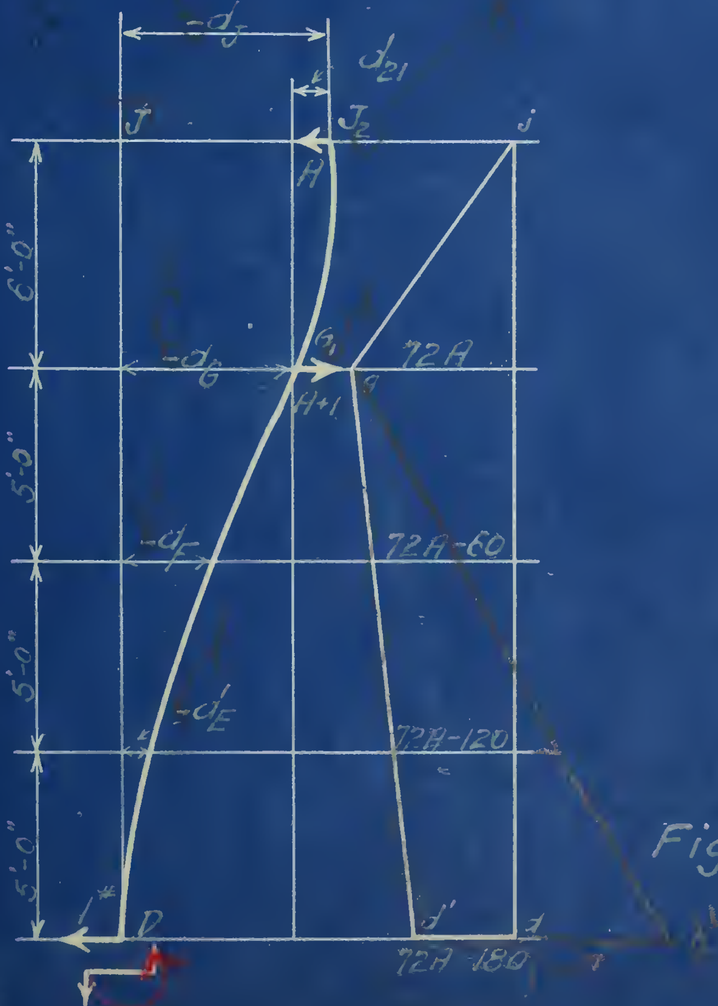

-dJ
d21
J
J2
j
H
6'-0"
-dG
72A
H+1
5'-0"
-dF
72A-60
5'-0"
-dE
72A-120
5'-0"
1*
D
d'
72A-180

Fig. 8.

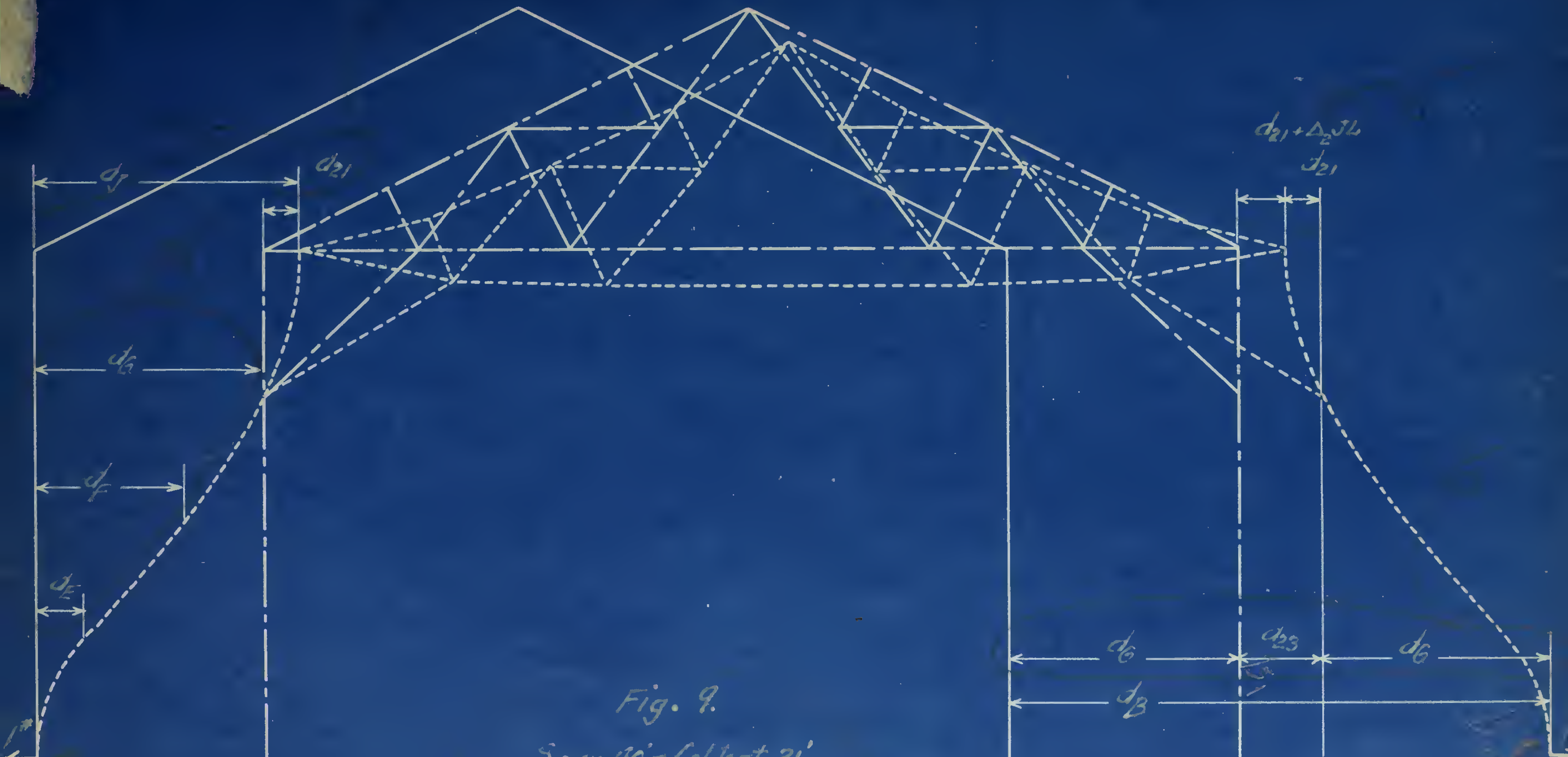


Fig. 9.

Span 40' - Col. hgt. 21'.
Class II - Case A.

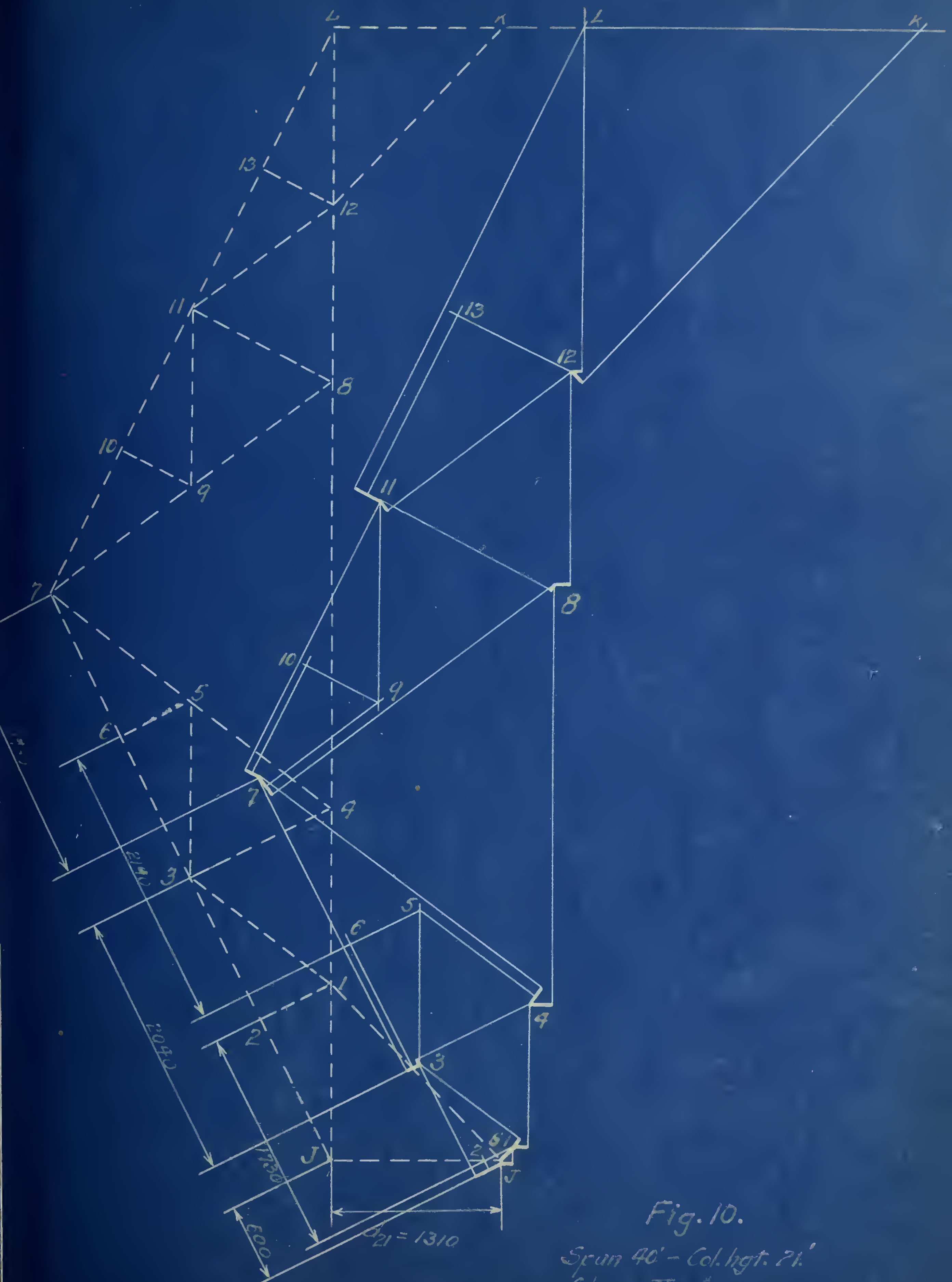
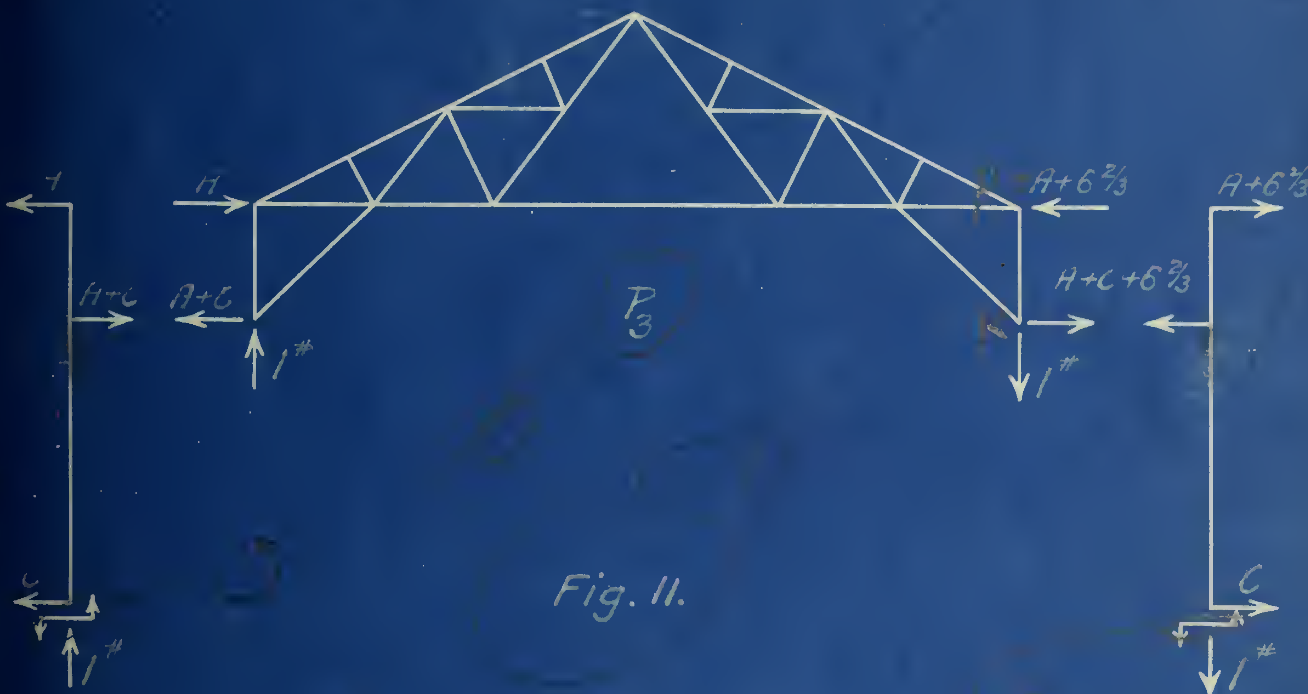


Fig. 10.

Span 40' - Col. hgt. 21'
Class II - Case A.



A
A
A+C
A+C
P_3
$1^{\#}$
$A+6\frac{2}{3}$
$A+6\frac{2}{3}$
$A+C+6\frac{2}{3}$
$1^{\#}$
C
C
$1^{\#}$
$1^{\#}$

Fig. 11.

d_L
a_1
$A+6\frac{2}{3}$
b_2
$6'-0"$
d_K
K_2
k
72A+480
$A+C+6\frac{2}{3}$
$15'-0"$
r_d
C
b'
b
72A+480-180C

Fig. 12.

Fig. 13.

Span 40' - Col. hgt. 21'.
Class II - Case B.

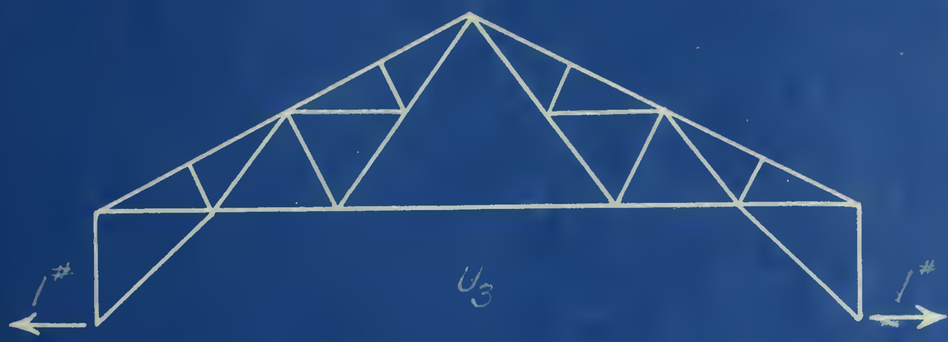


Fig. 14.

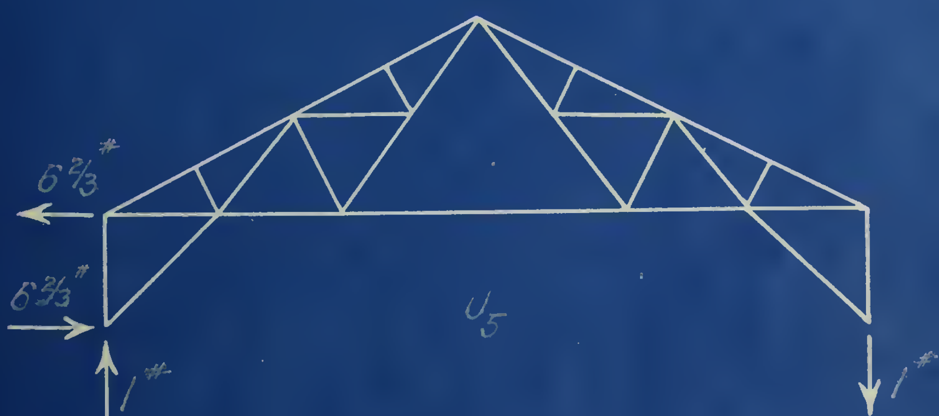


Fig. 15.

Fig. 16.

Span 40' - Col. hgt. 21.'
Class II - Case B.

A
A+C
A+C
R
P4
A+6⅔R
A+6⅔R
A+C+6⅔R
R
C
C
R
R

Fig. 17.

d_2
d_k
Δ_1
Δ_2
Δ_3
Δ_4
A+6⅔R
A+C+6⅔R
K_2
6'-0"
15'-0"
1-180C

Fig. 18.

Fig. 19.

Span 40' - Col. hgt. 21'.
Class II - Case C.

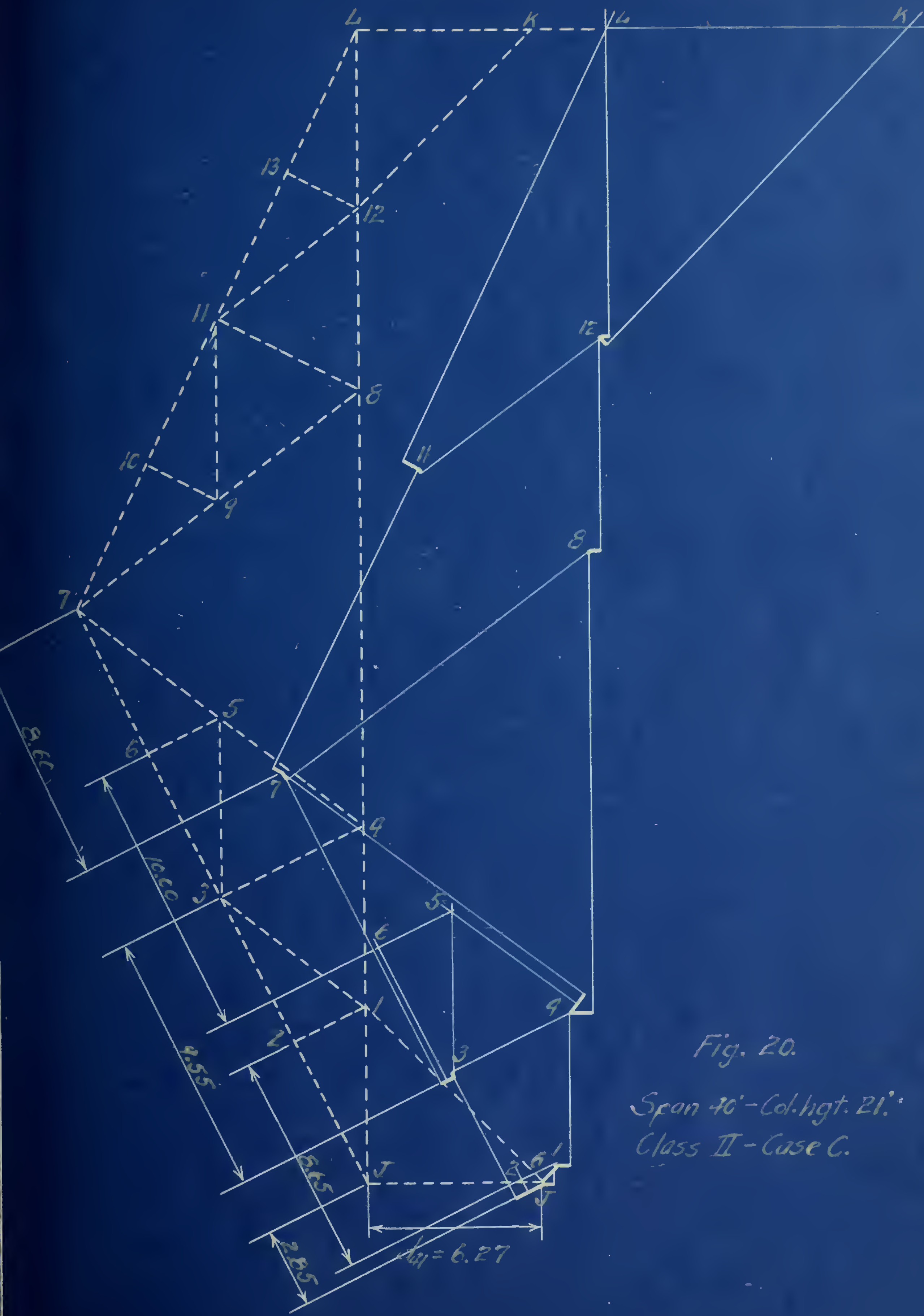


Fig. 20.

Span 40' - Col. hgt. 21'

Class II - Case C.

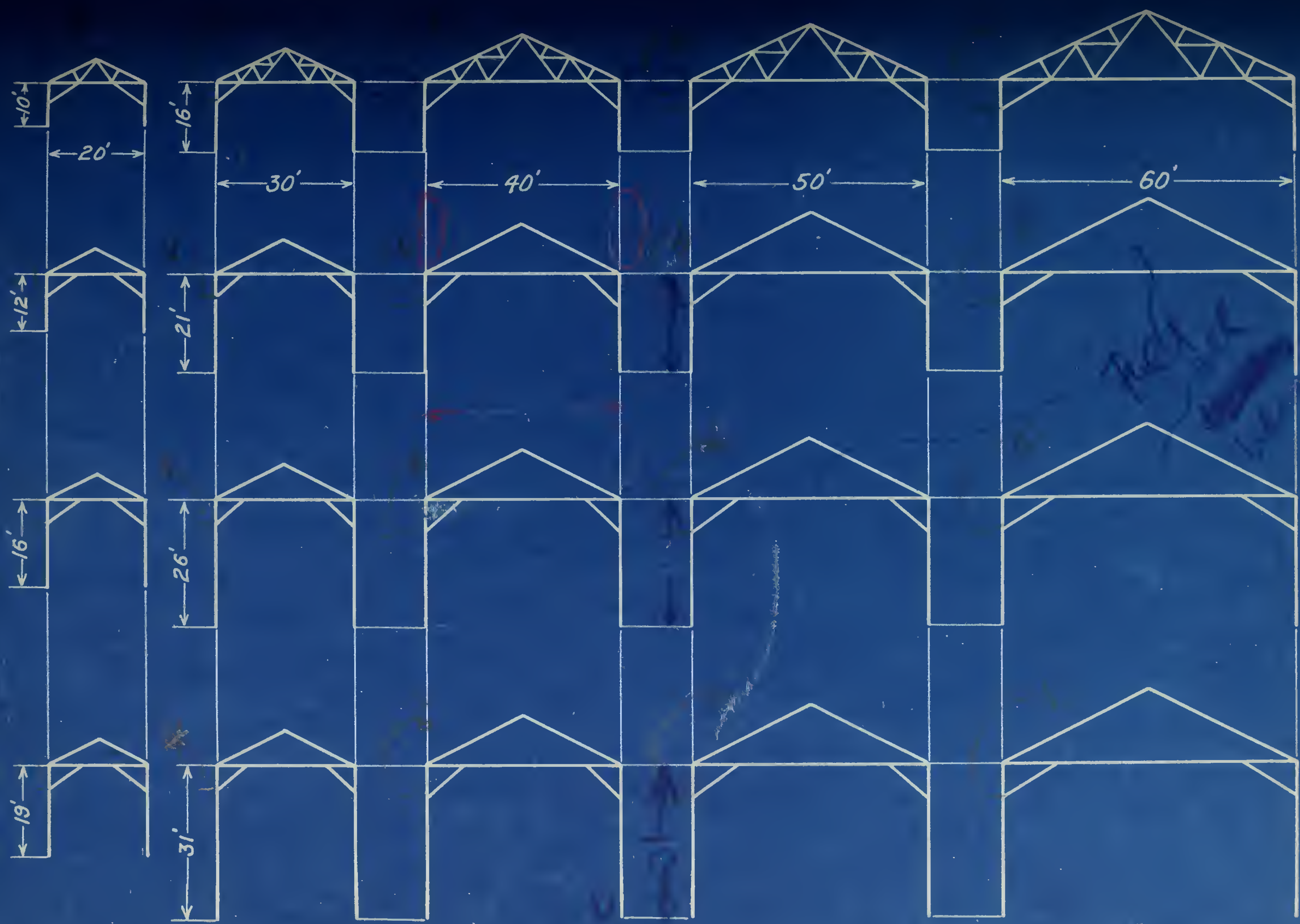


Fig. 21.

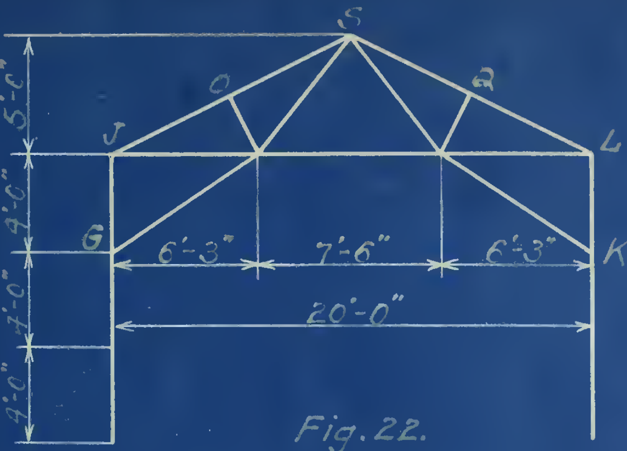


Fig. 22.

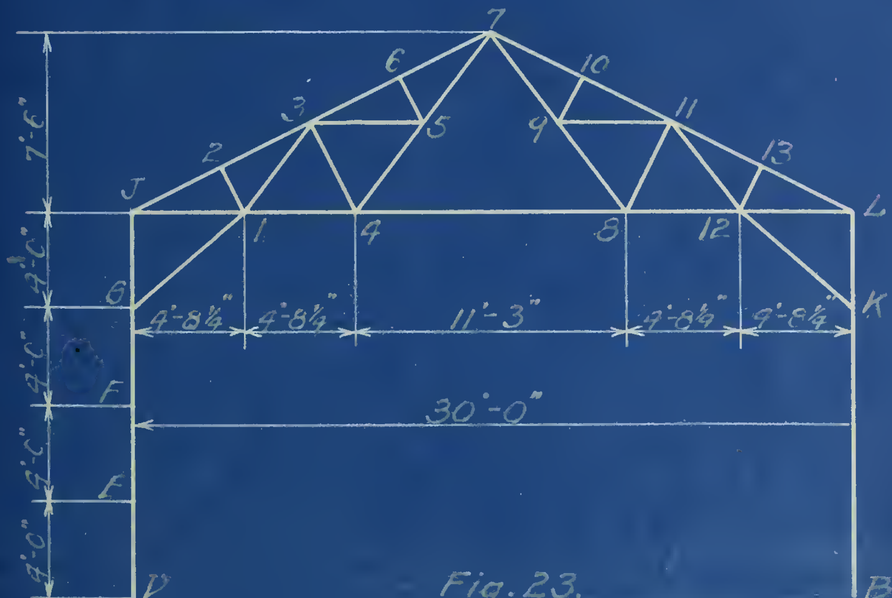


Fig. 23.

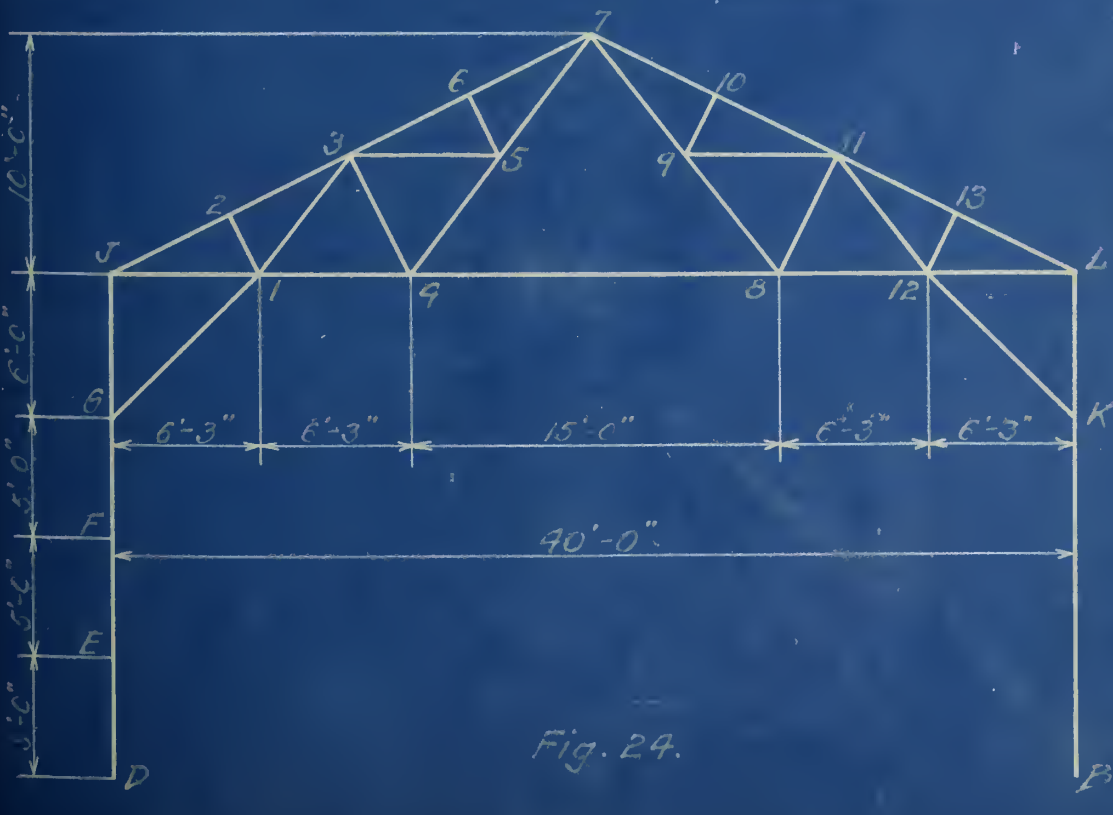
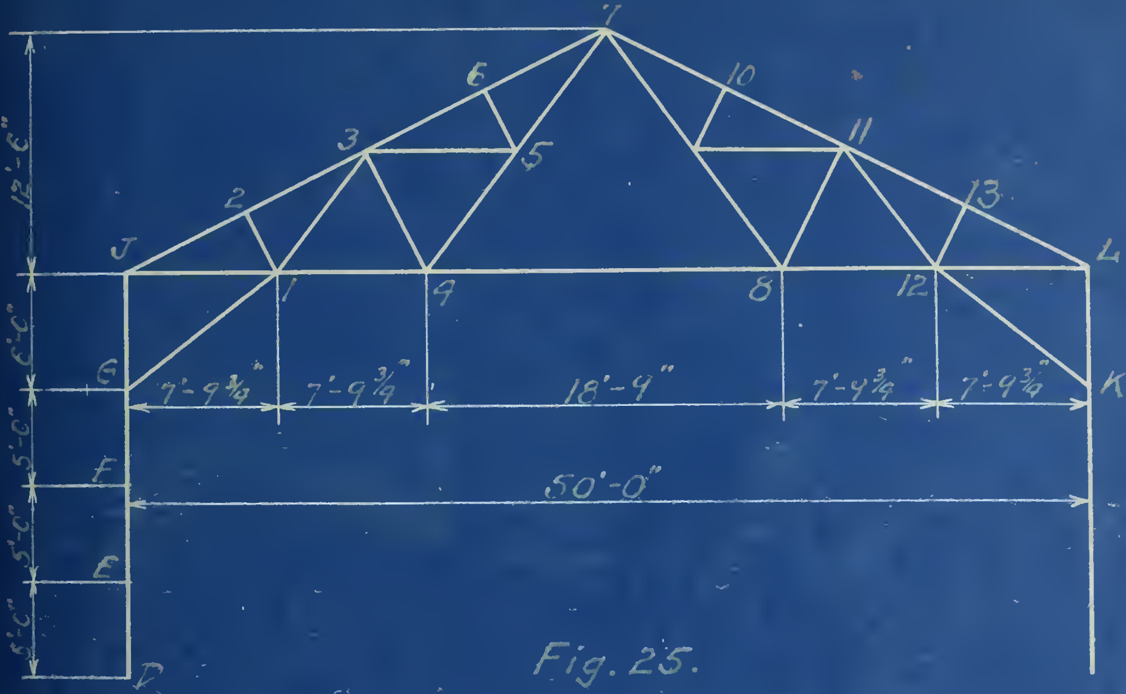
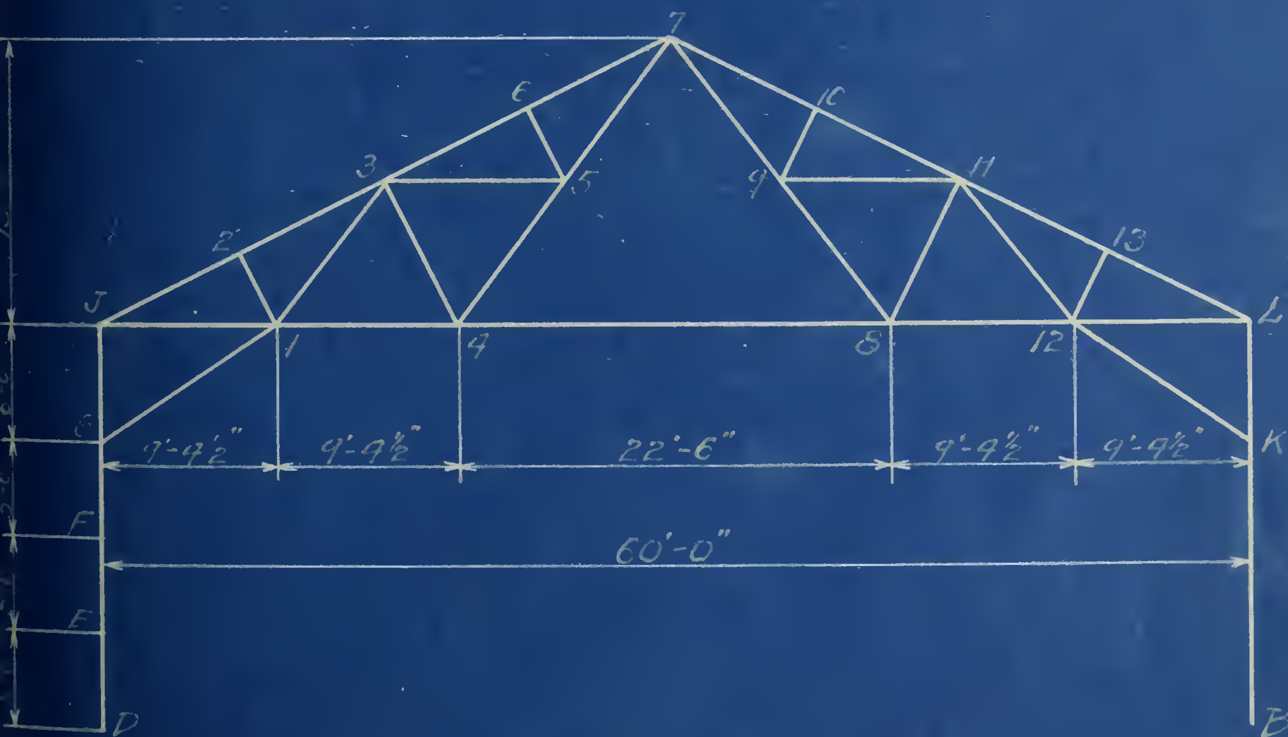


Fig. 24.



7
6
10
3
5
11
2
13
J
L
1
4
8
12
G
K
F
E
D
12'-6"
6'-0"
5'-0"
5'-0"
5'-0"
7'-9¾"
7'-9¾"
18'-4"
7'-9¾"
7'-9¾"
50'-0"

Fig. 25.



7
6
10
3
5
9
11
2
13
J
L
1
4
8
12
K
F
E
D
B
9'-9½"
9'-9½"
22'-6"
9'-9½"
9'-9½"
60'-0"

Fig. 26.

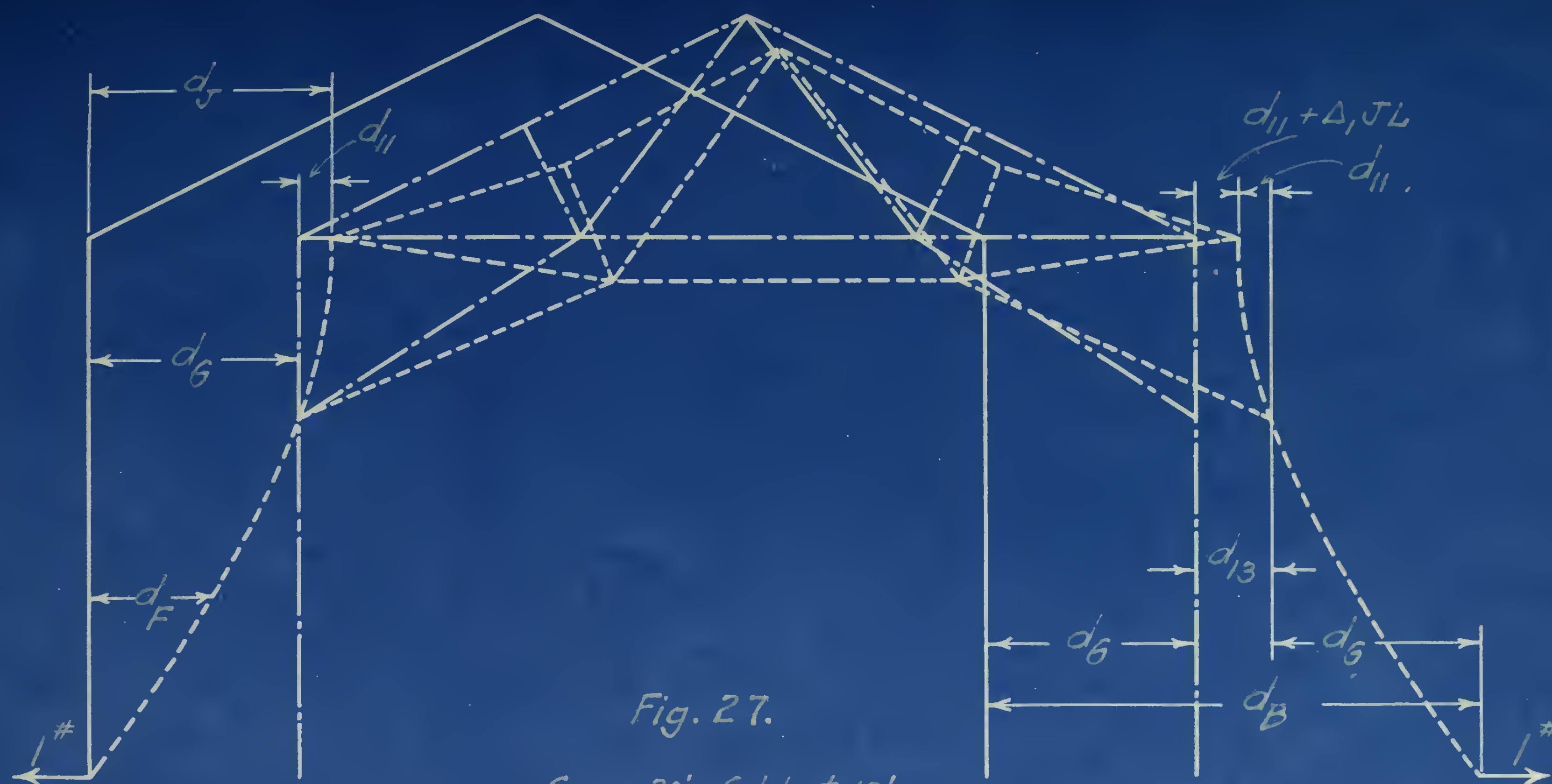


Fig. 27.

Span 20'-Col. hgt. 12'.
Class I - Case A.

Fig. 28.

Span 20'-Col. hgt. 10'.
Class I-Case A.

Fig. 29.

Span 20'-Col. hgt. 12'
Class I-Case A.

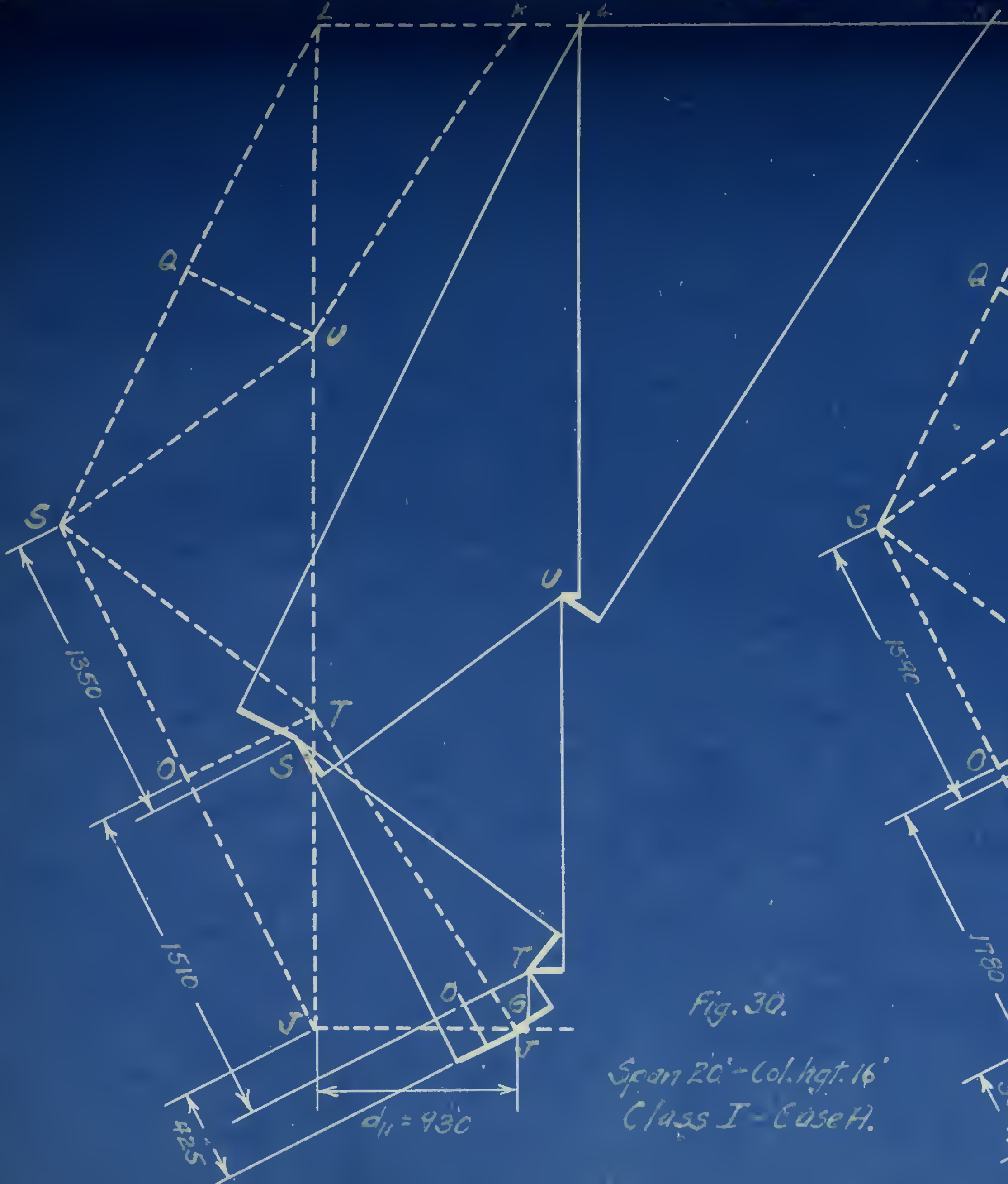


Fig. 30.

Span 20'–Col. hgt. 16'
Class I – Case A.

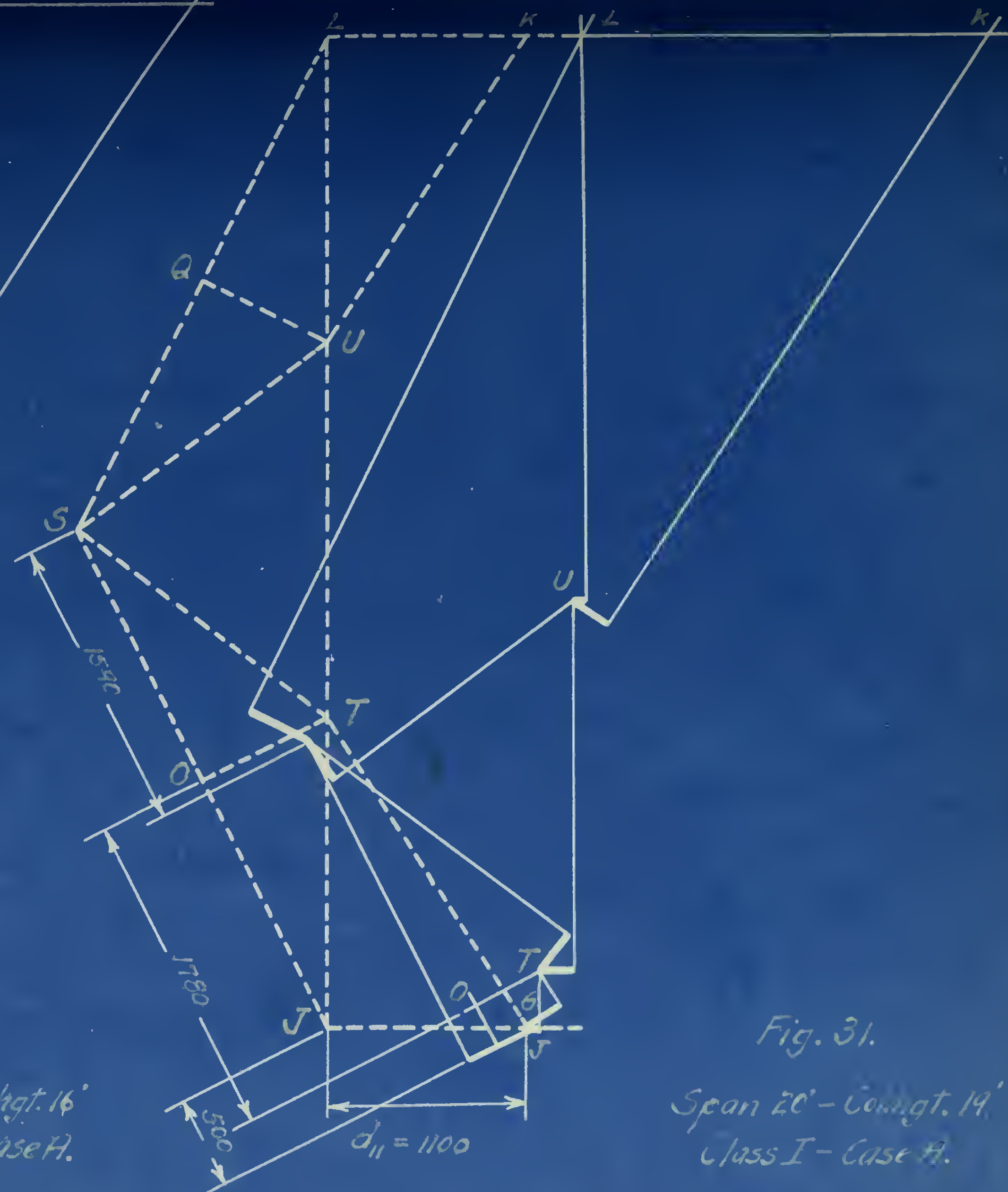


Fig. 31.

Span 20'–Col. hgt. 19'
Class I – Case A.

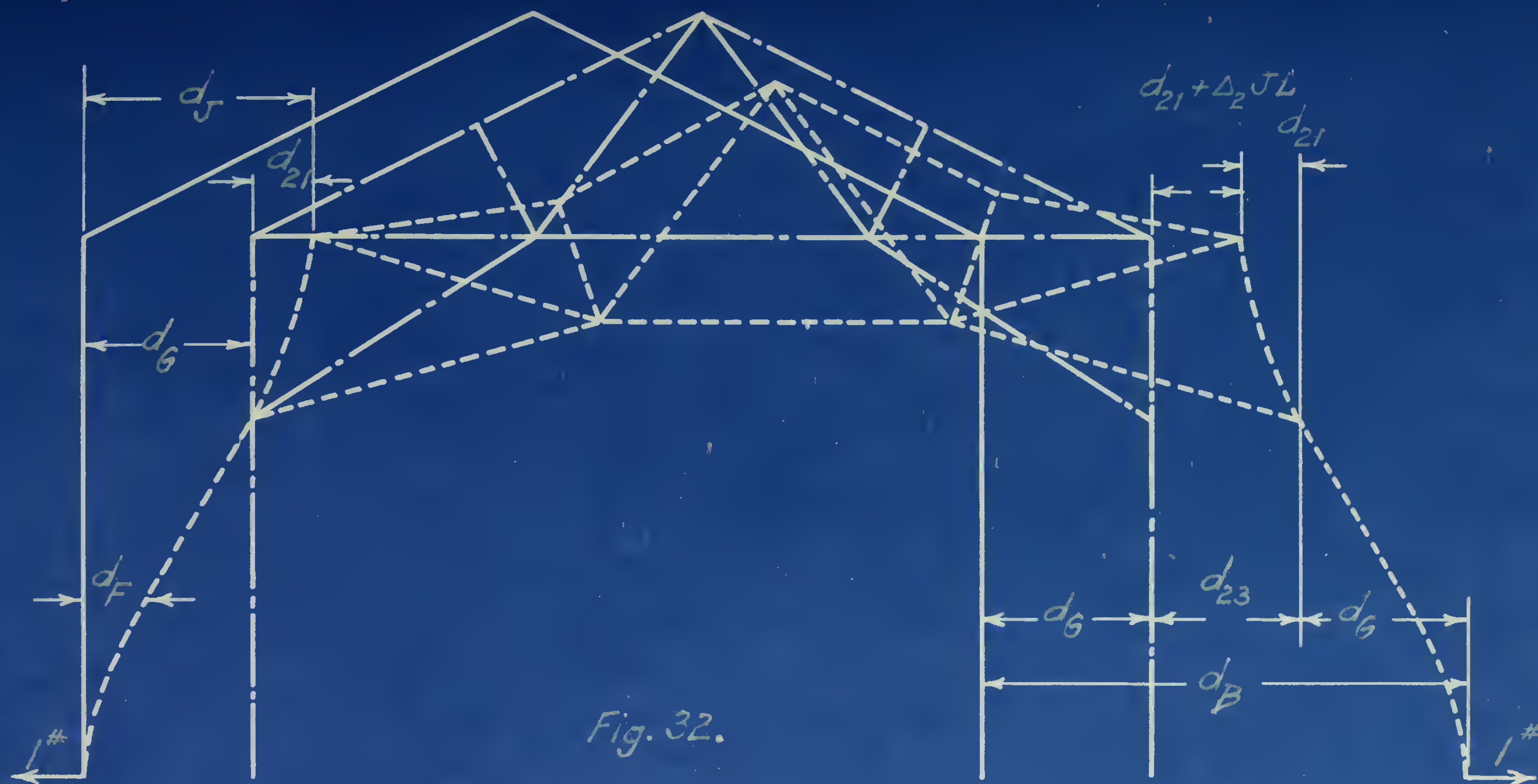


Fig. 32.

Span 20'-Col. hgt. 12'.
Class II-Case A.

Fig. 33.

Span 20'-Col. hgt. 10'.
Class II - Case A.

Fig. 34.

Span 20'-Col. hgt. 12'.
Class II - Case A.

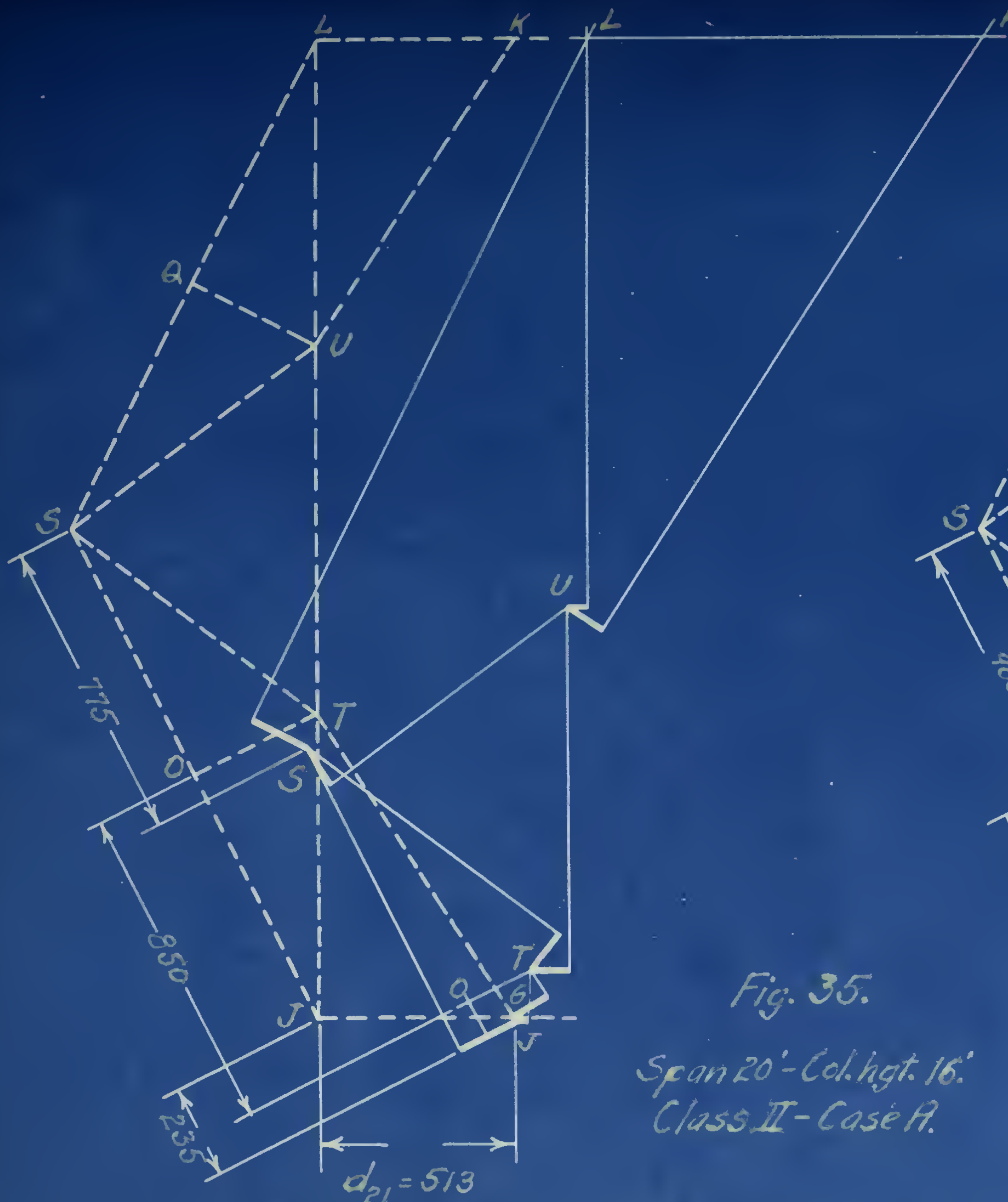


Fig. 35.

Span 20'-Col. hgt. 16'.
Class II - Case A.

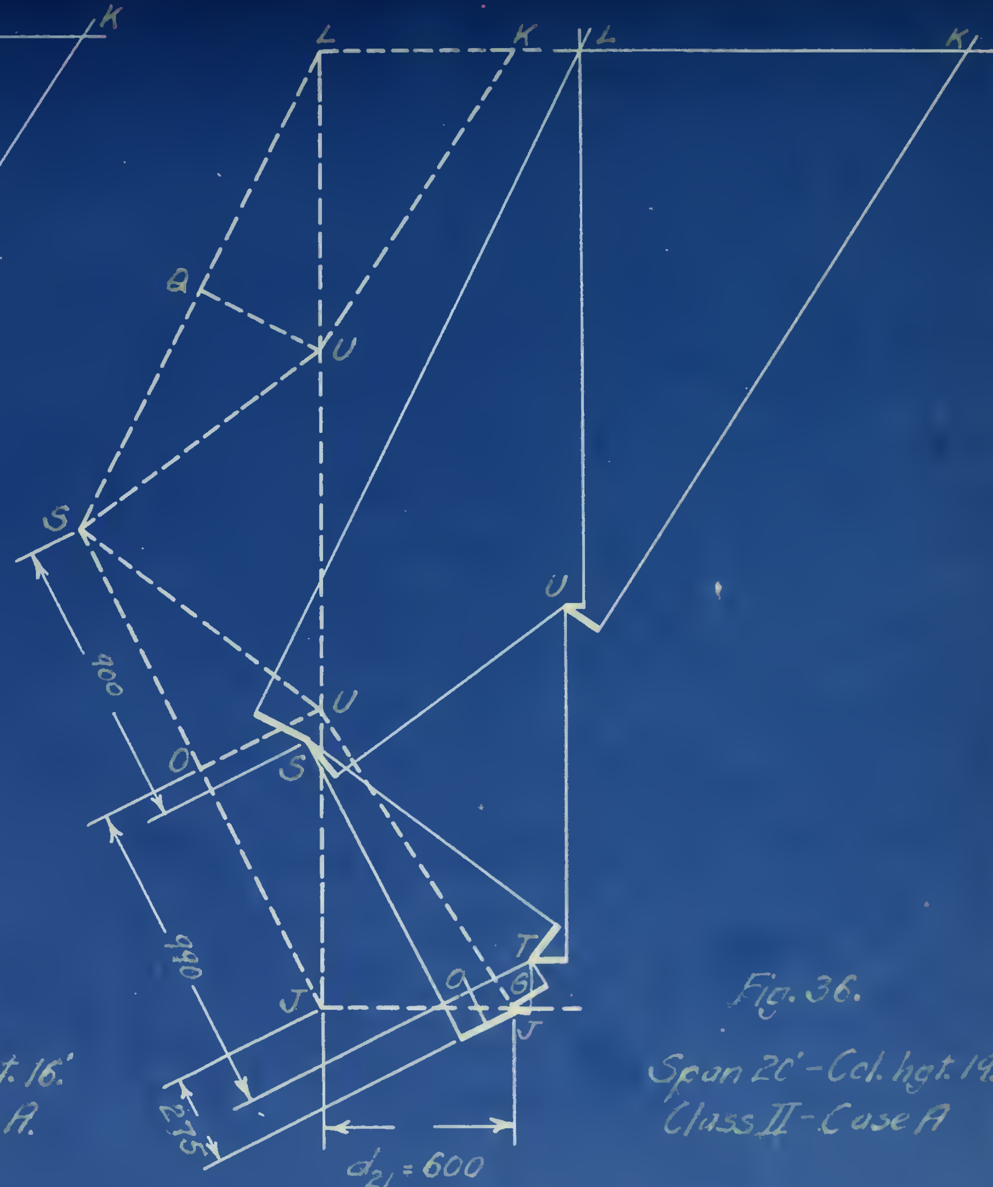


Fig. 36.

Span 20'-Col. hgt. 19.
Class II - Case A

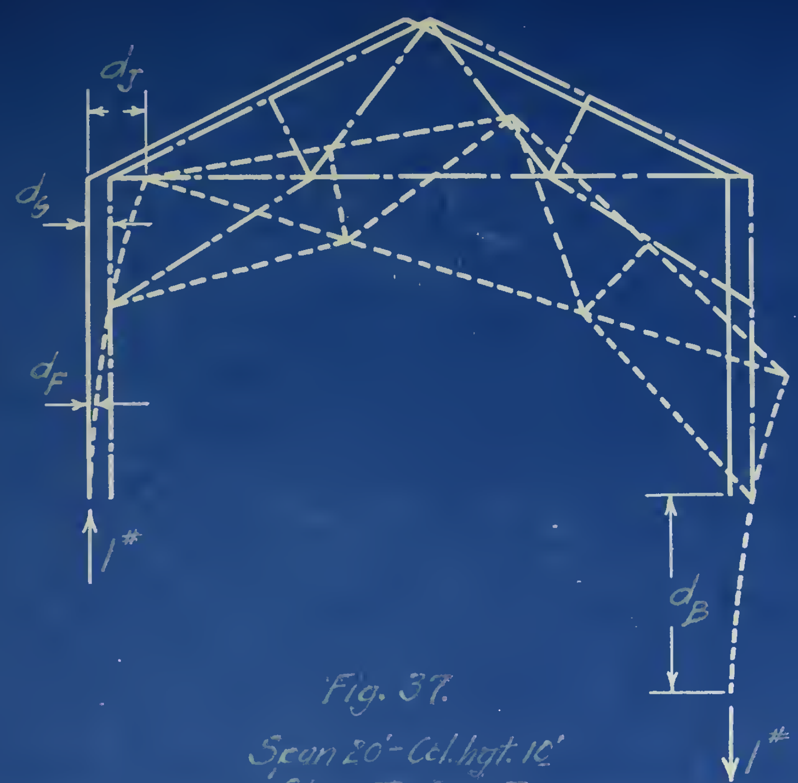


Fig. 37.

Span 20'-Col. hgt. 10'
Class II-Case B.

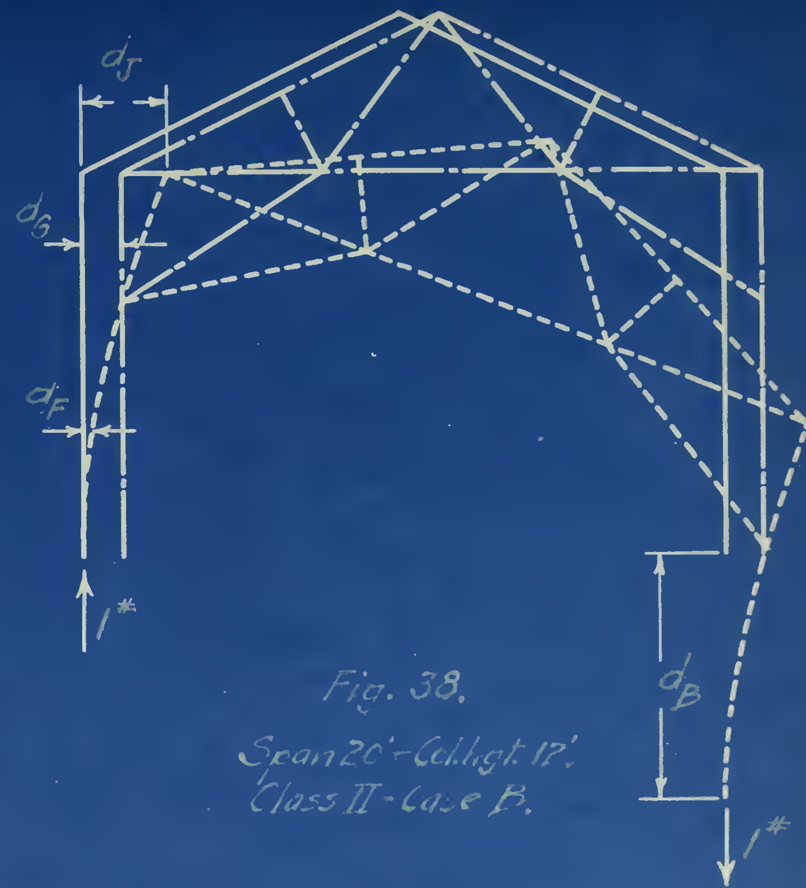


Fig. 38.

Span 20'-Col. hgt. 12'
Class II-Case B.

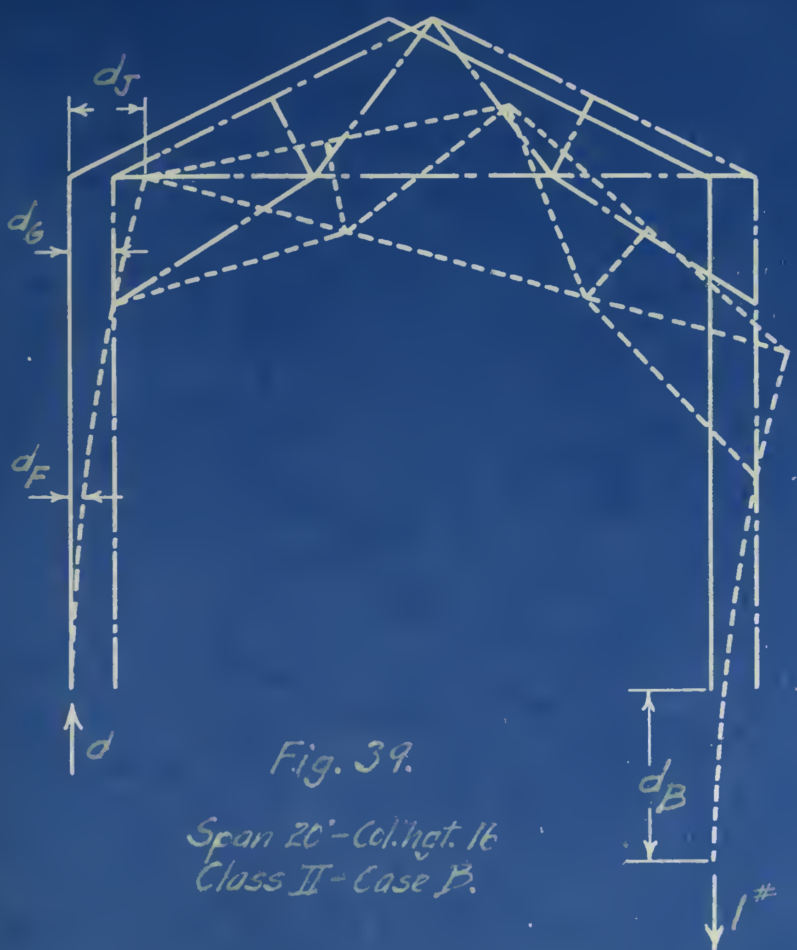


Fig. 39.

Span 20' - Col. hgt. 16
Class II - Case B.

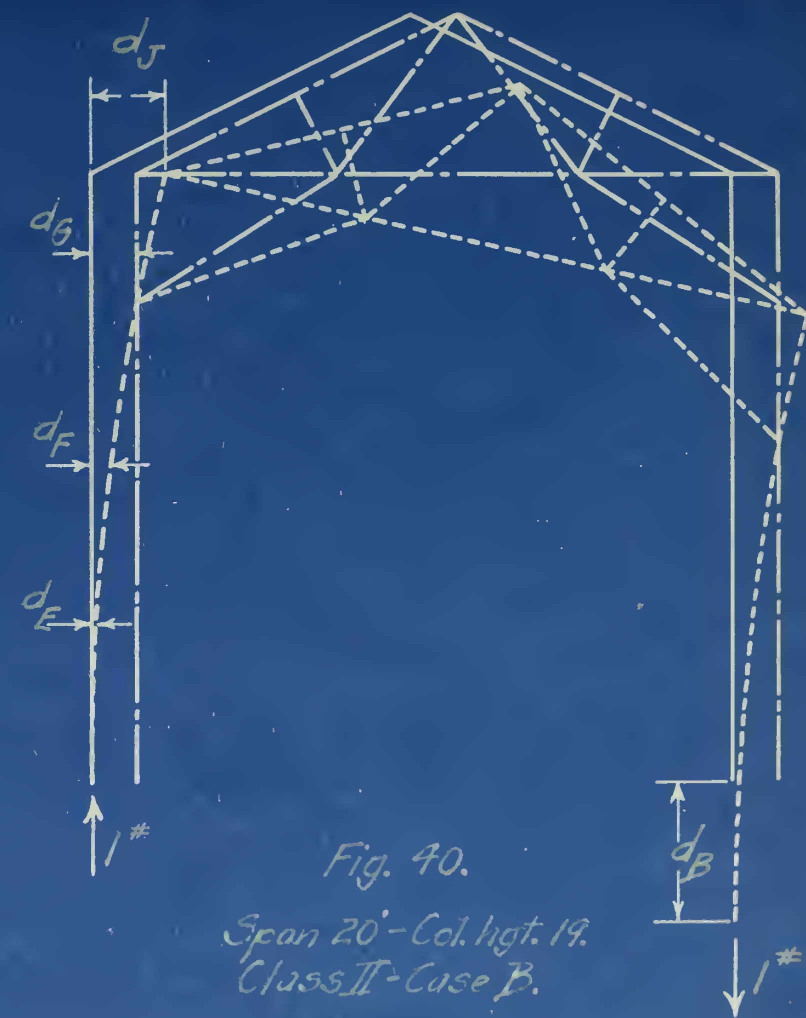


Fig. 40.

Span 20' - Col. hgt. 19.
Class II - Case B.

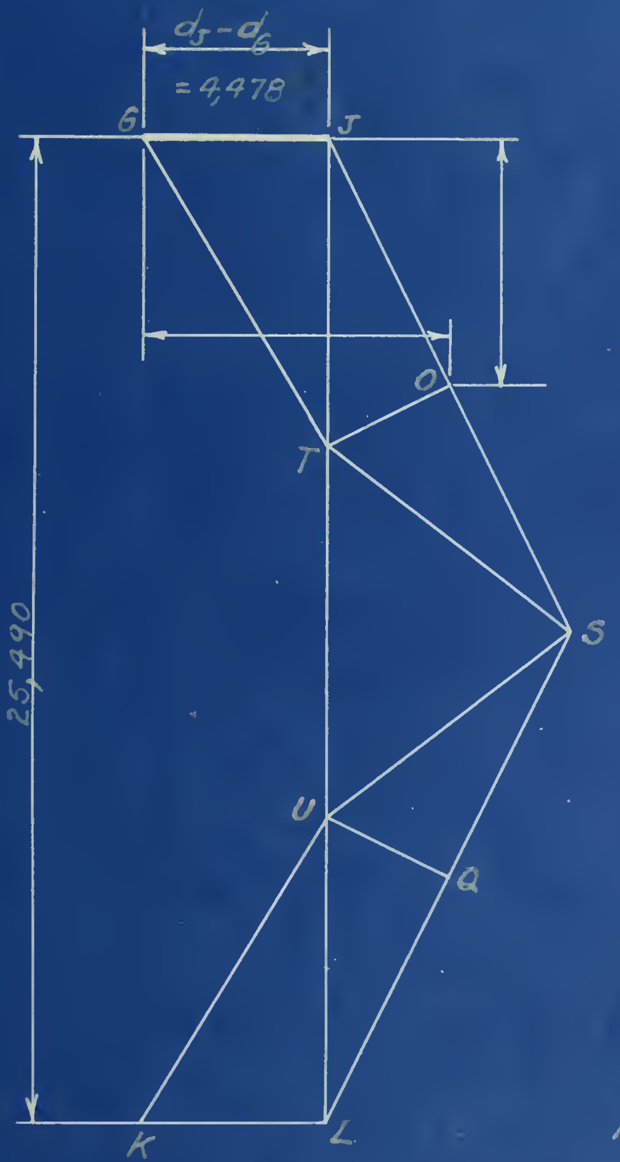


Fig. 41.

Span 20' - Col. hgt. 12.
Class II - Case B.

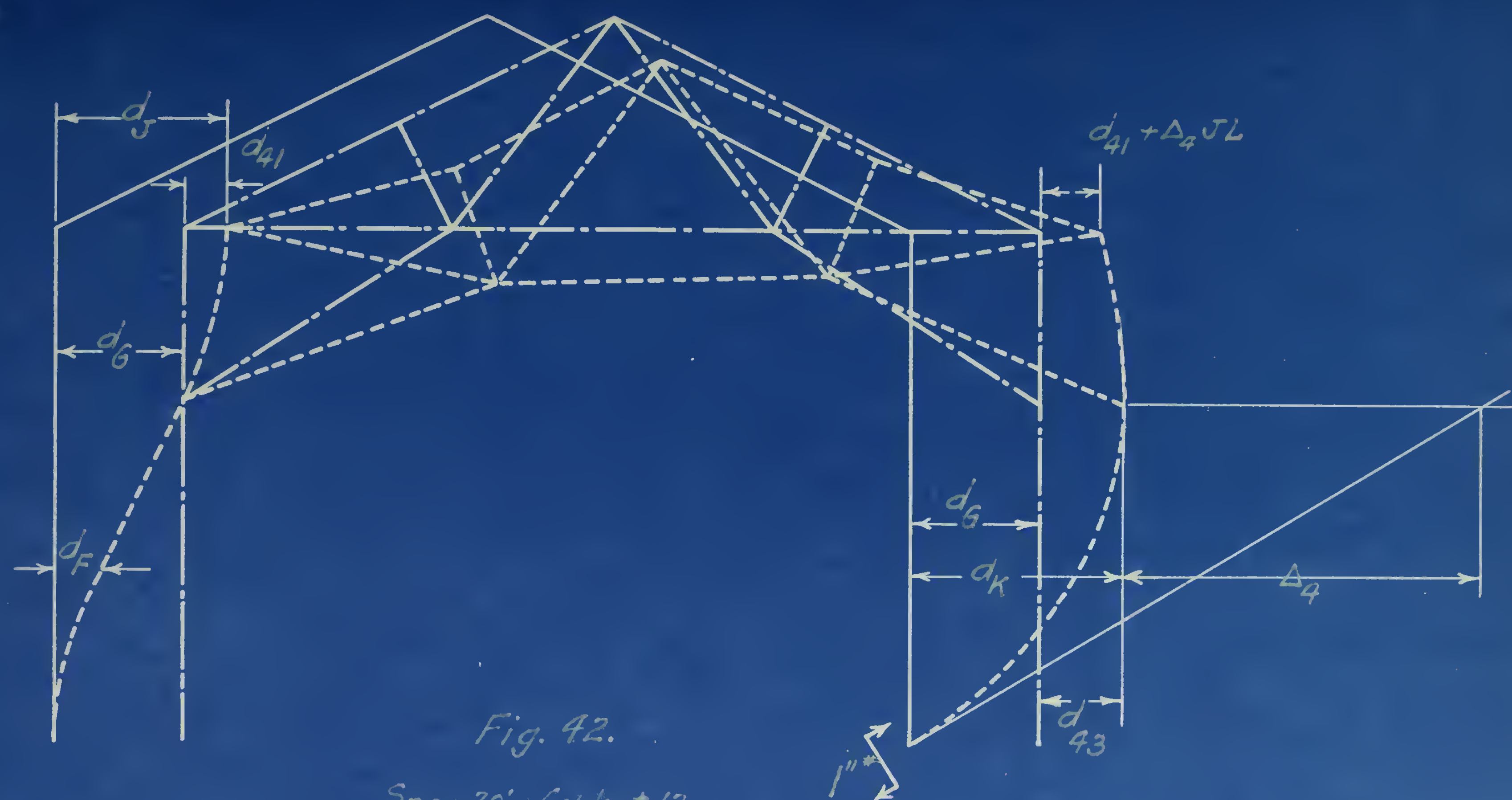


Fig. 42.

Span 20' - Col. hgt. 12.
Class II - Case C.

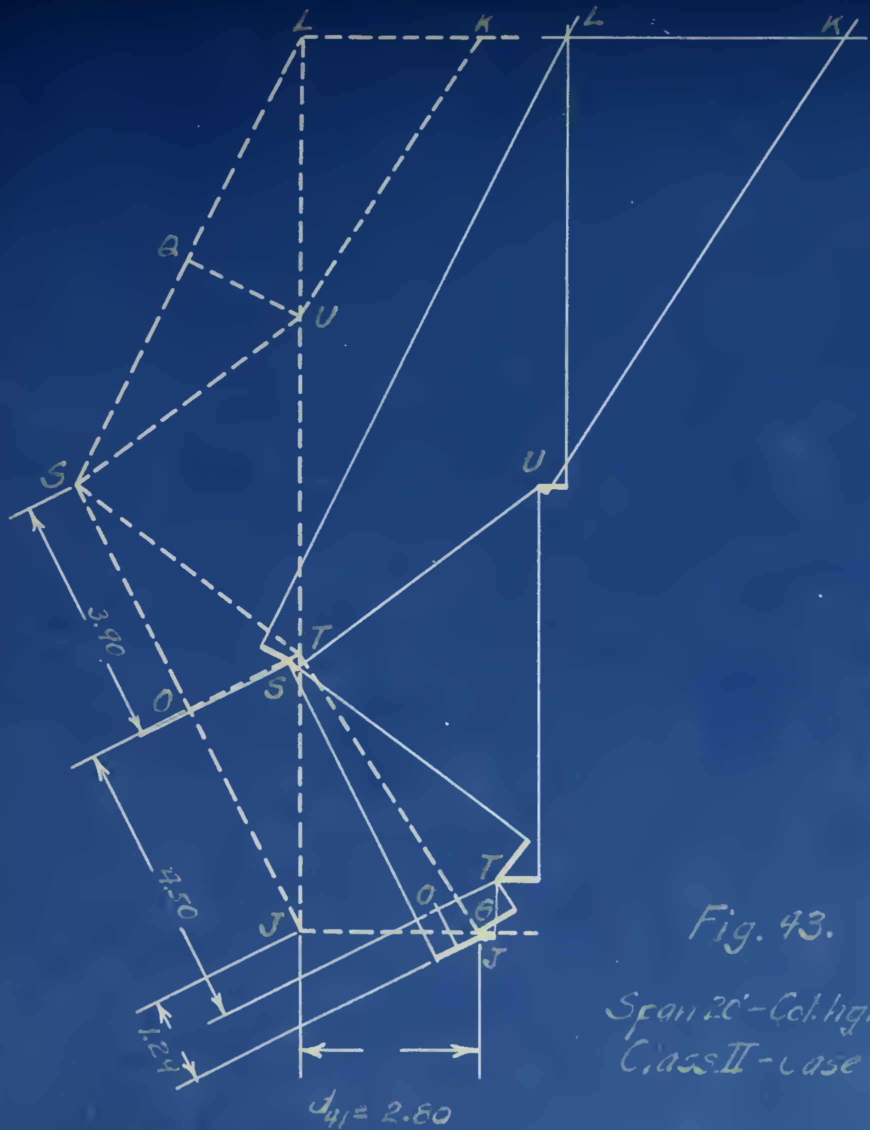


Fig. 43.

Span 20' – Col. hgt. 10'.
Class II – Case C.

Fig. 44.

Span 20' – Col. hgt. 12'
Class II – Case C.

Fig. 45.

Span 20' – Col. hgt. 16'.
Class II – Case C.

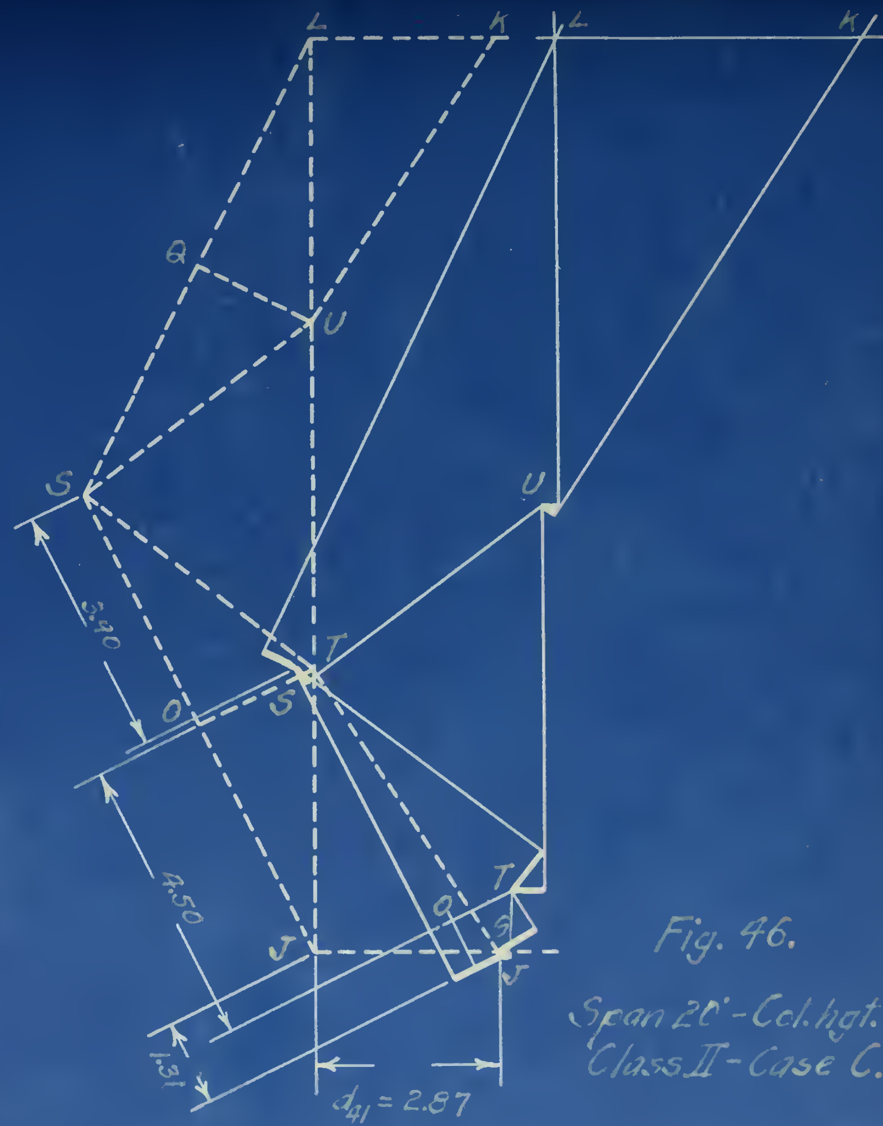


Fig. 46.

Span 20' – Col. hgt. 19'.
Class II – Case C.

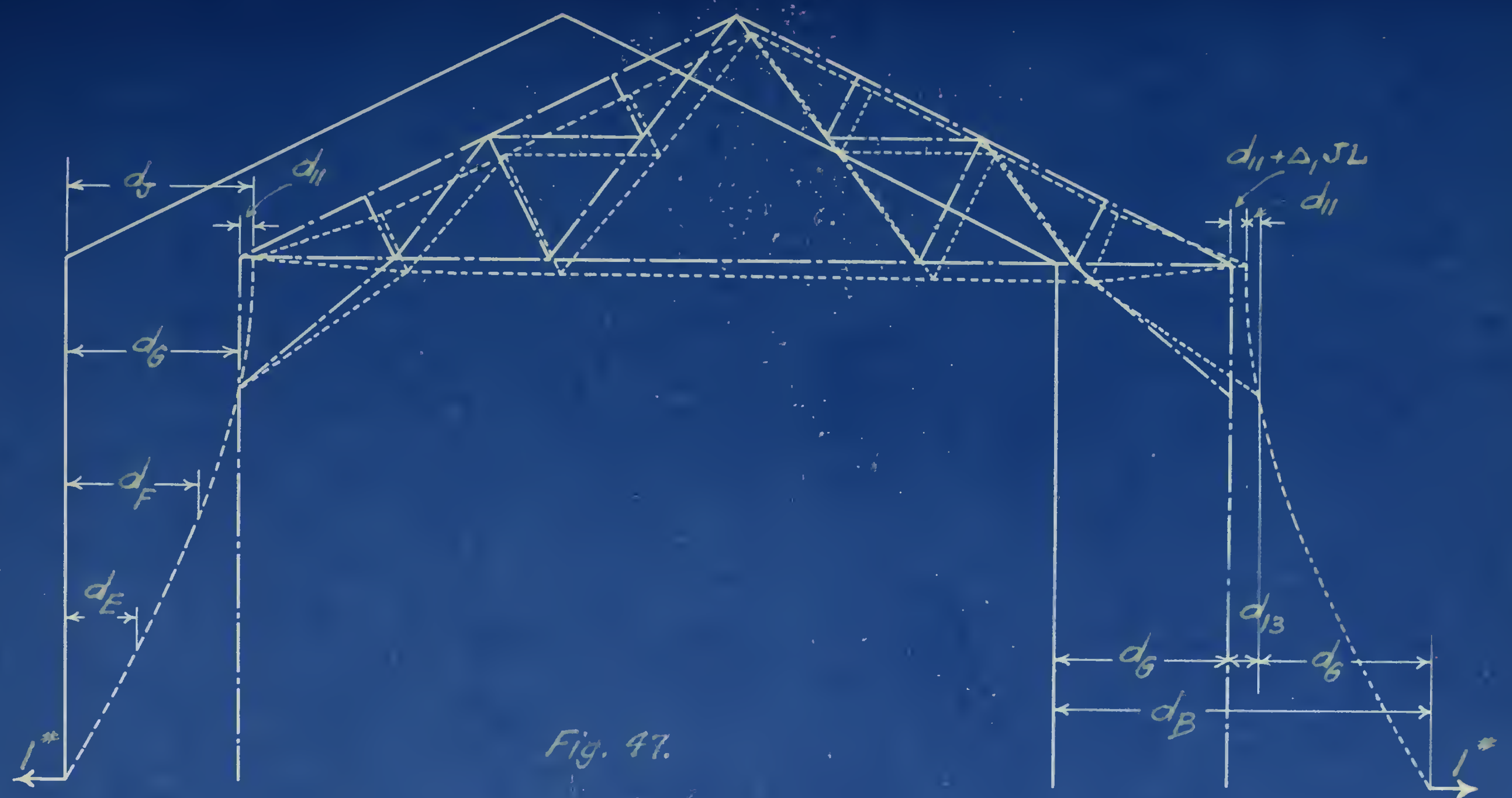


Fig. 47.

Span 30'-Col. hgt. 16.
Class I-Case A.

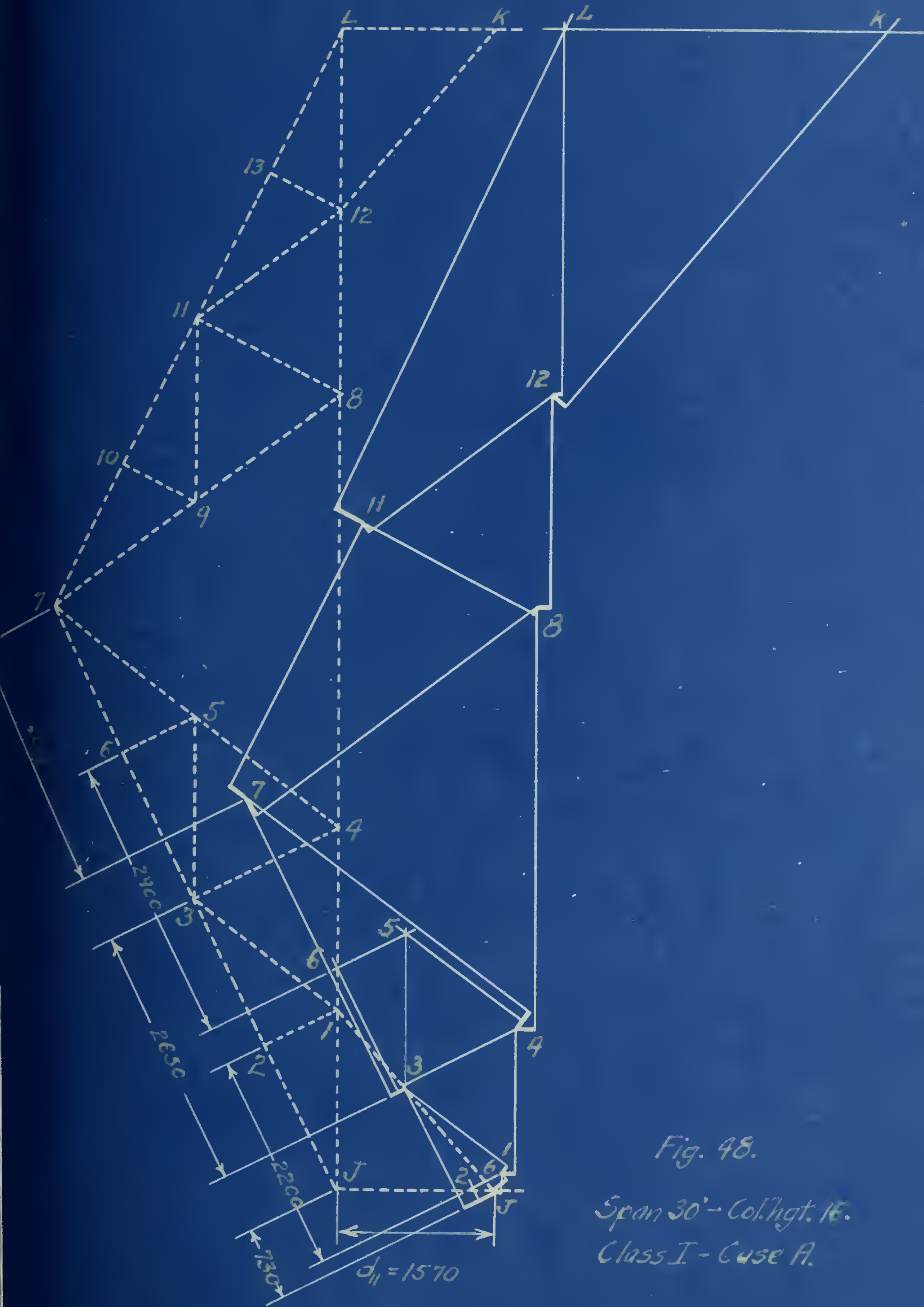


Fig. 48.

Span 30' - Col. hgt. 16.
Class I - Case A.

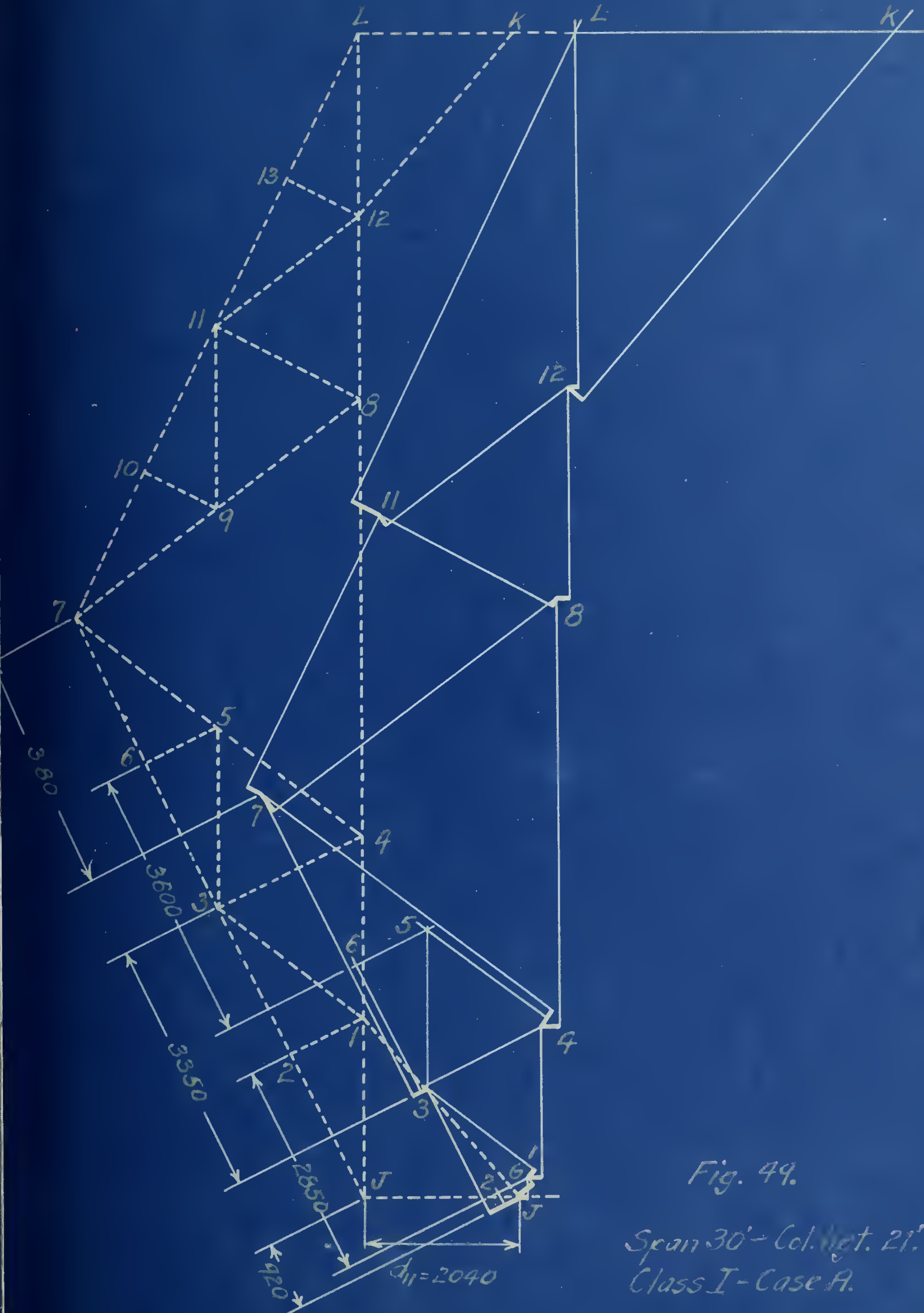


Fig. 49.

Span 30' - Col. Hgt. 21'.
Class I - Case A.

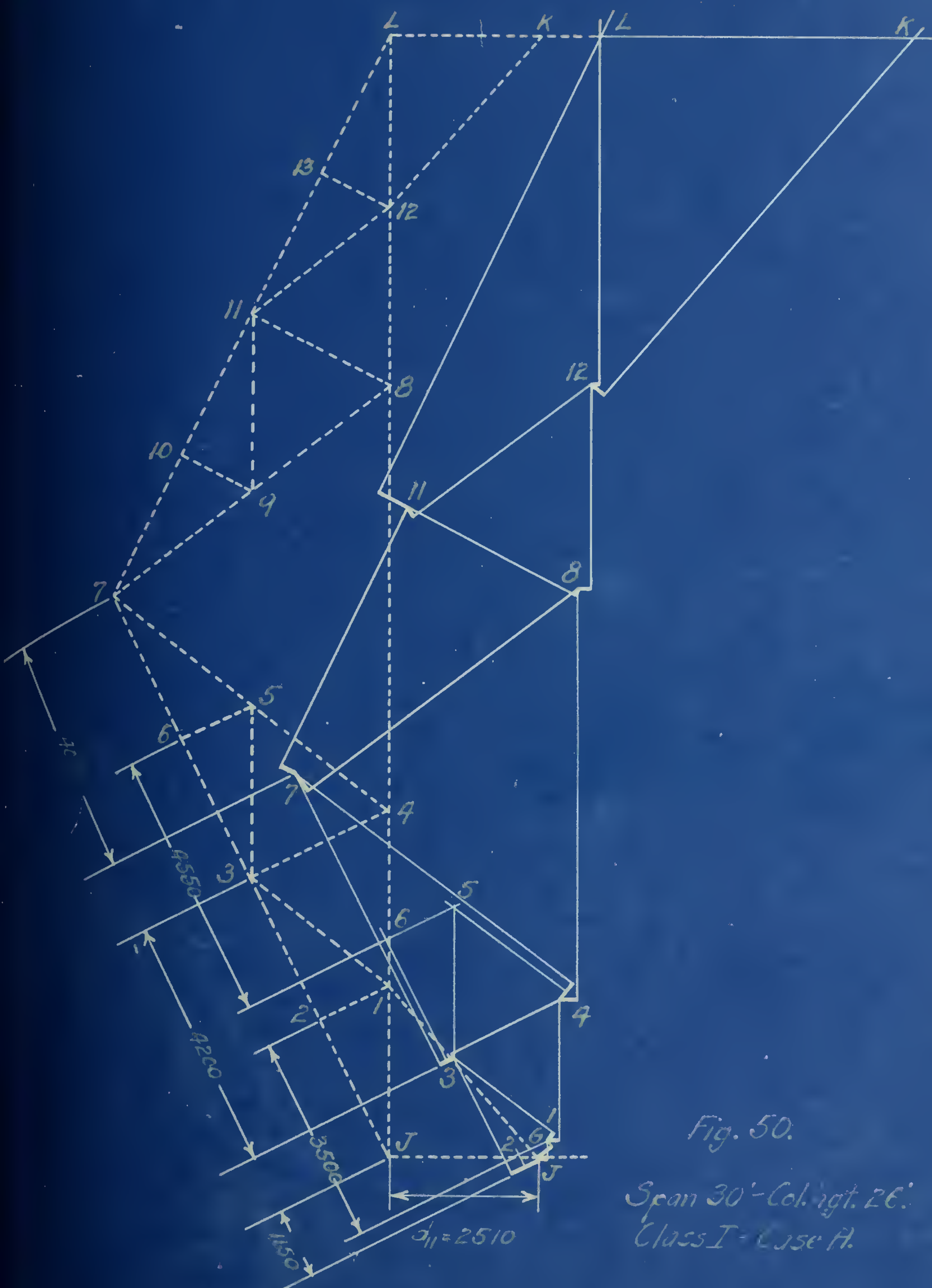


Fig. 50.

Span 30'- Col. lgt. 26'.
Class I - Case A.

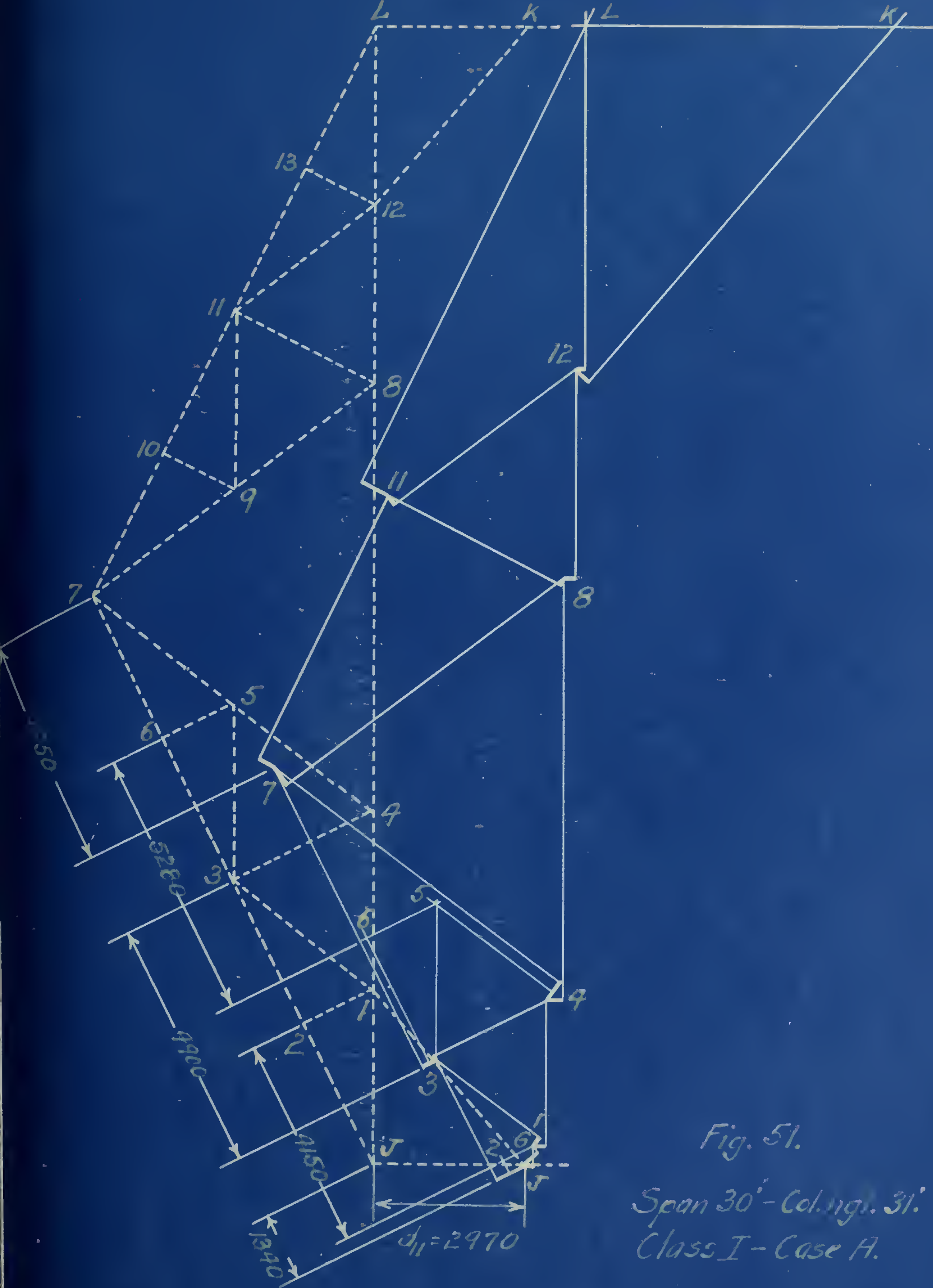


Fig. 51.

Span 30' - Col. lgt. 31'.
Class I - Case A.

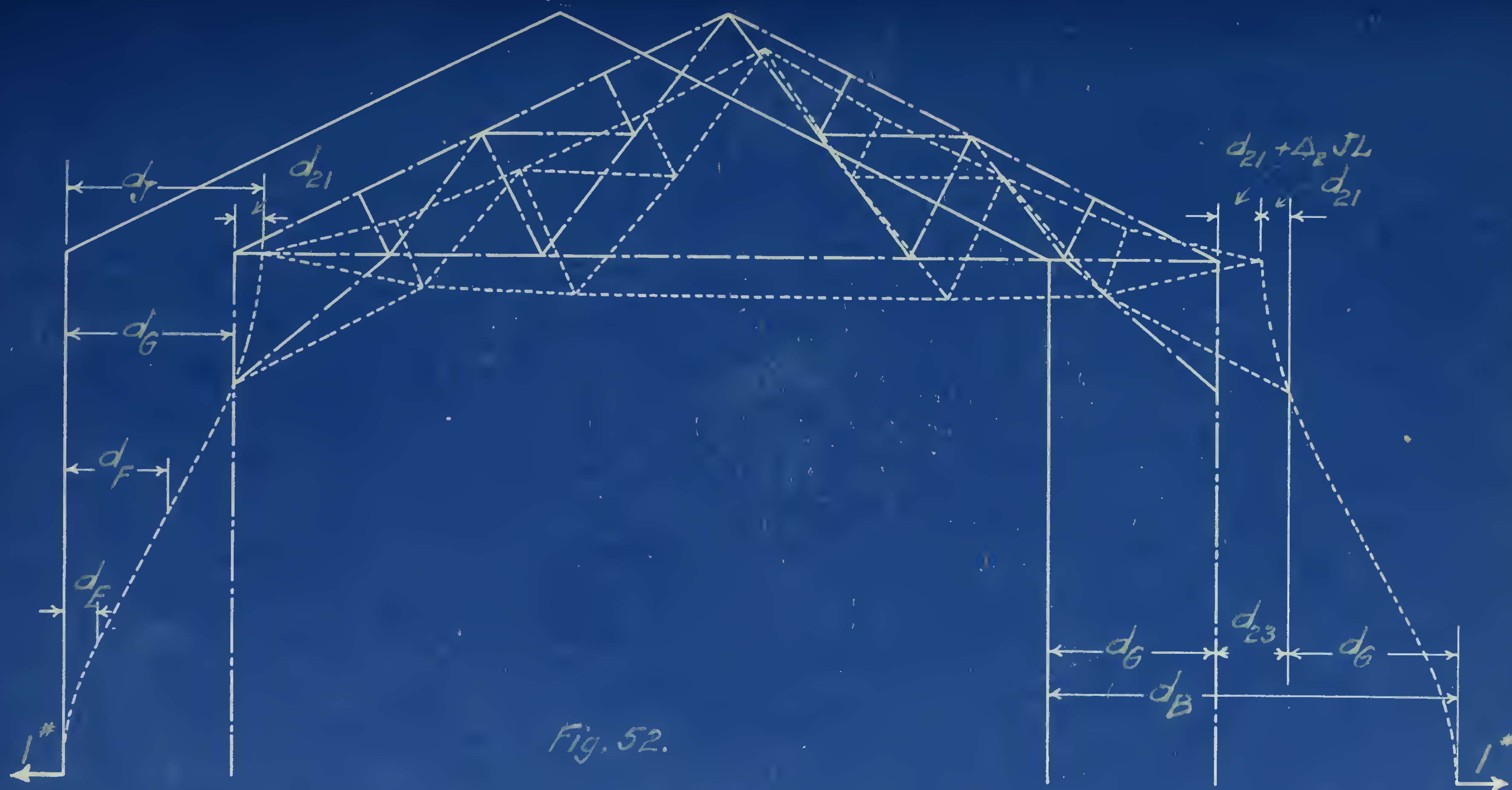


Fig. 52.

Span 30' - Col. hgt. 16'.
Class II - Case A.

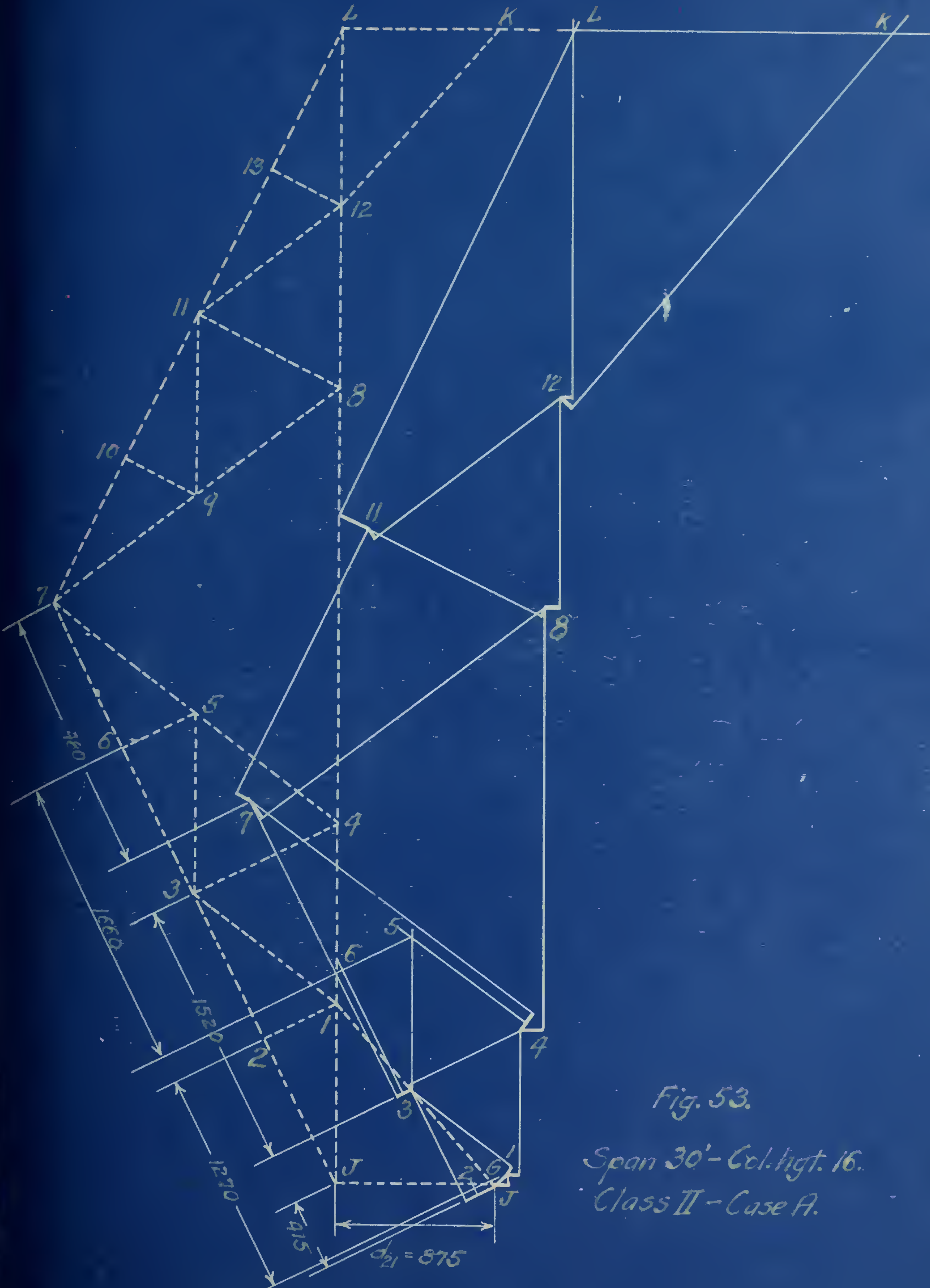


Fig. 53.

Span 30' - Col. hgt. 16.
Class II - Case A.

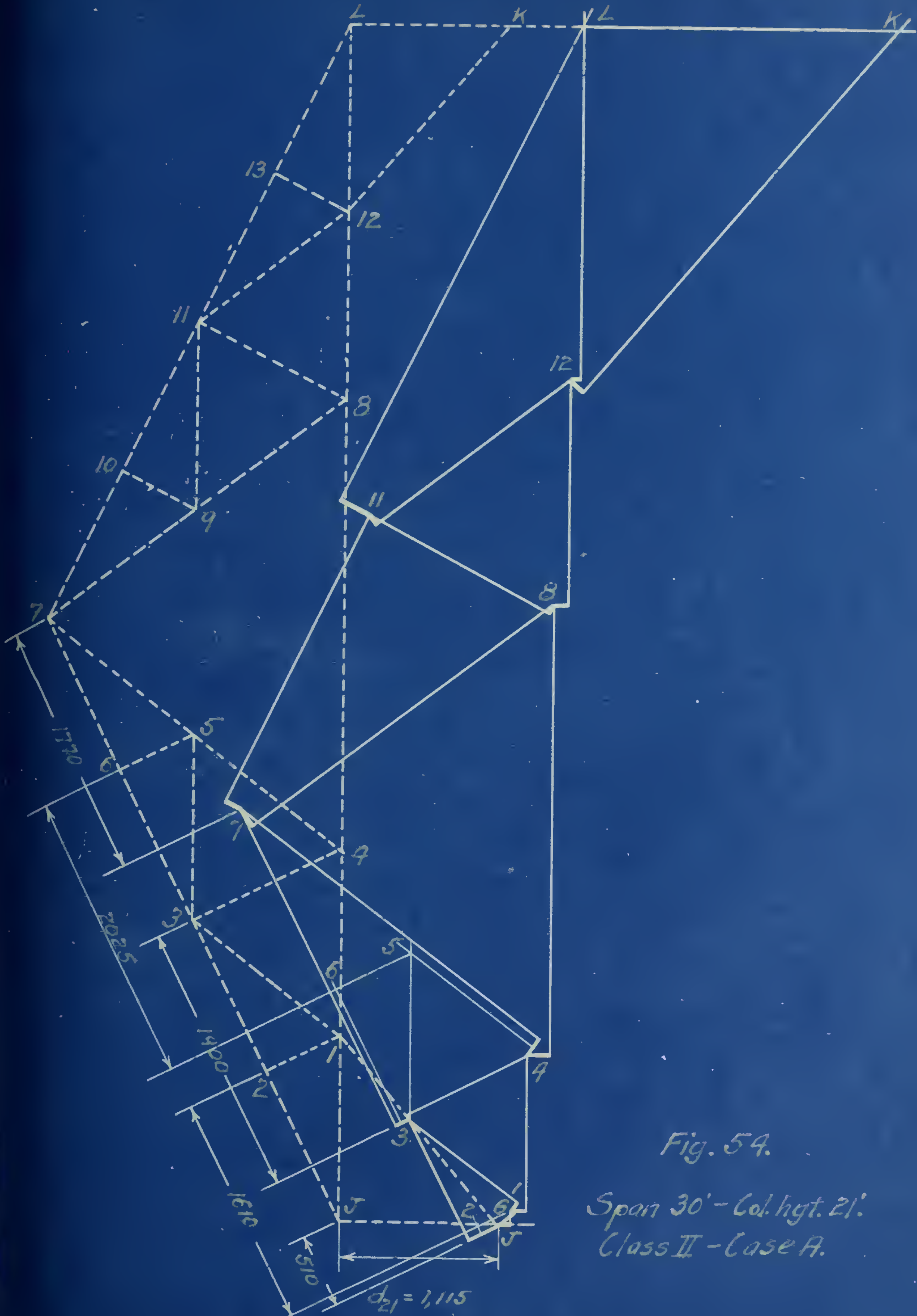


Fig. 54.

Span 30' – Col. hgt. 21'.
Class II – Case A.

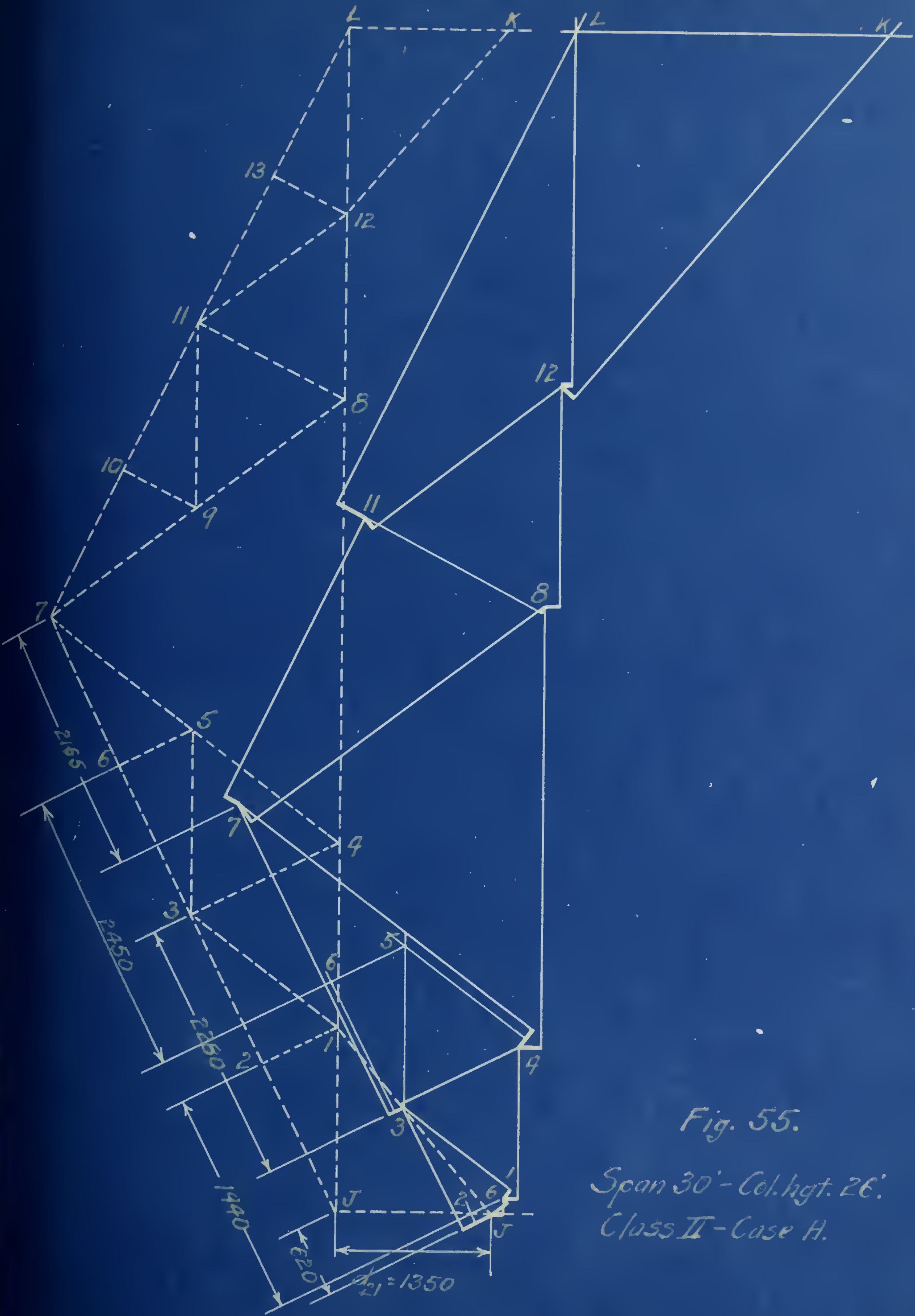


Fig. 55.

Span 30' - Col. hgt. 26'.
Class II - Case H.

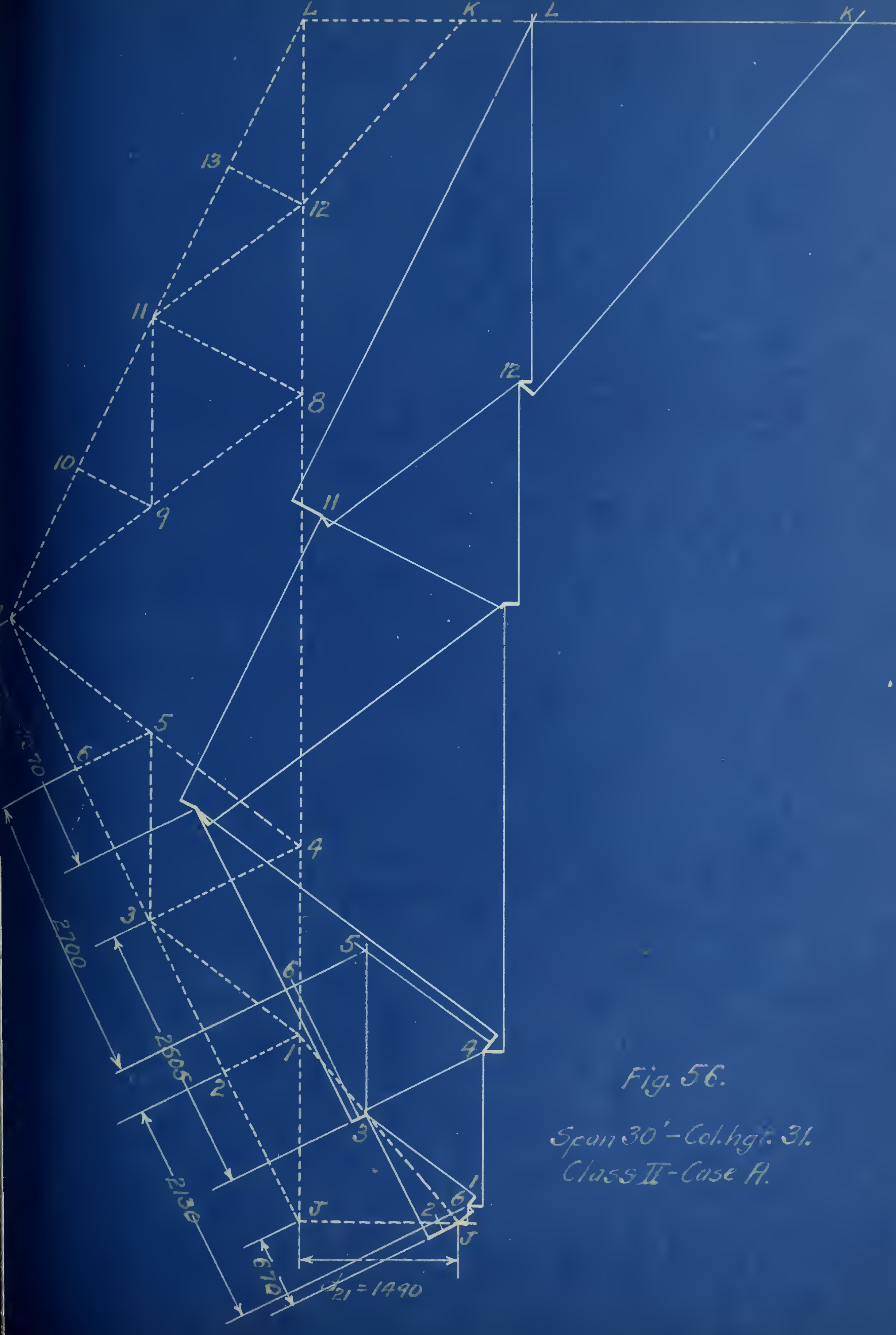


Fig. 56.

Span 30' - Col. hgt. 31.
Class II - Case A.

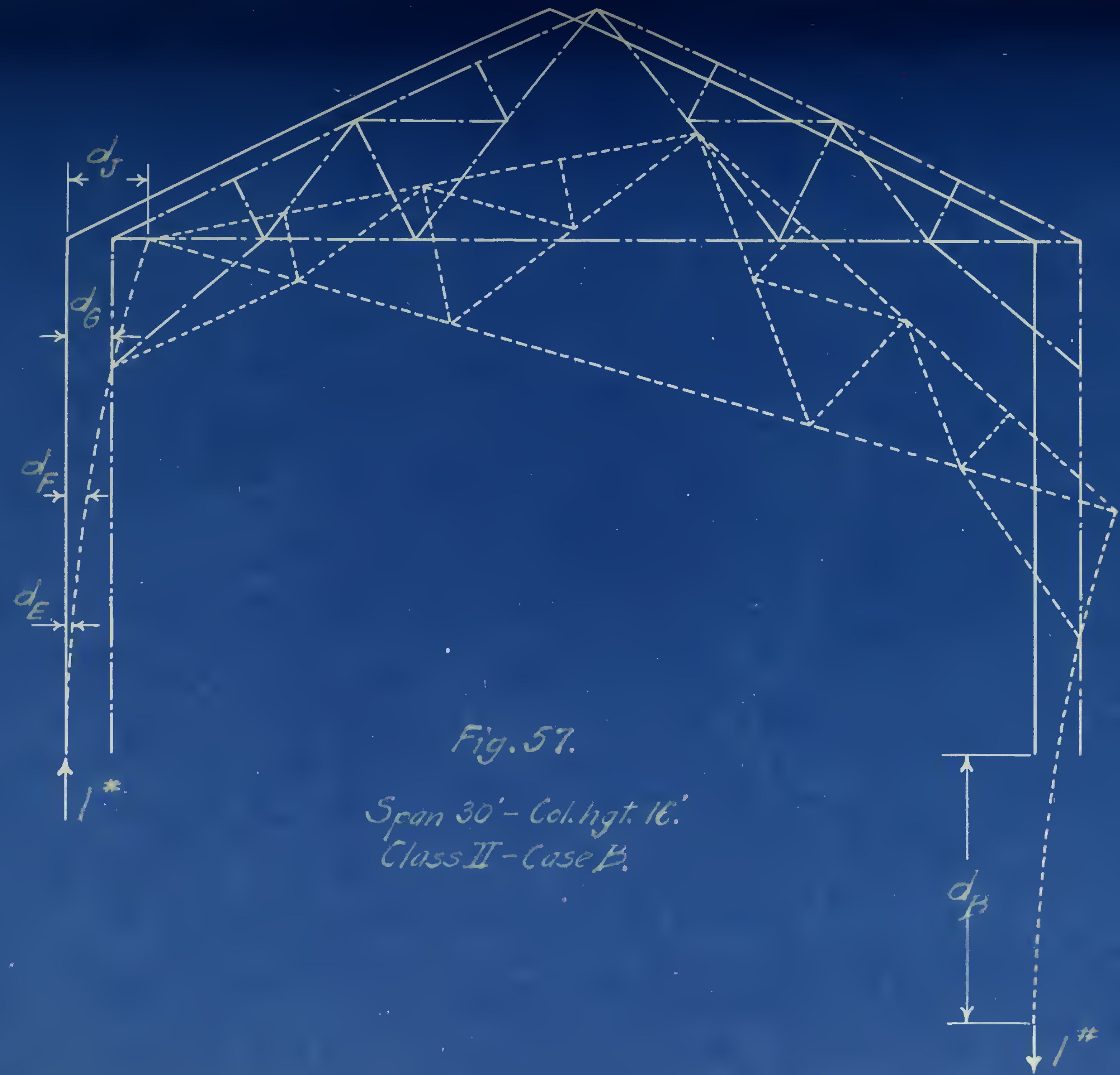


Fig. 57.

Span 30' - Col. hgt. 16'.
Class II - Case B.

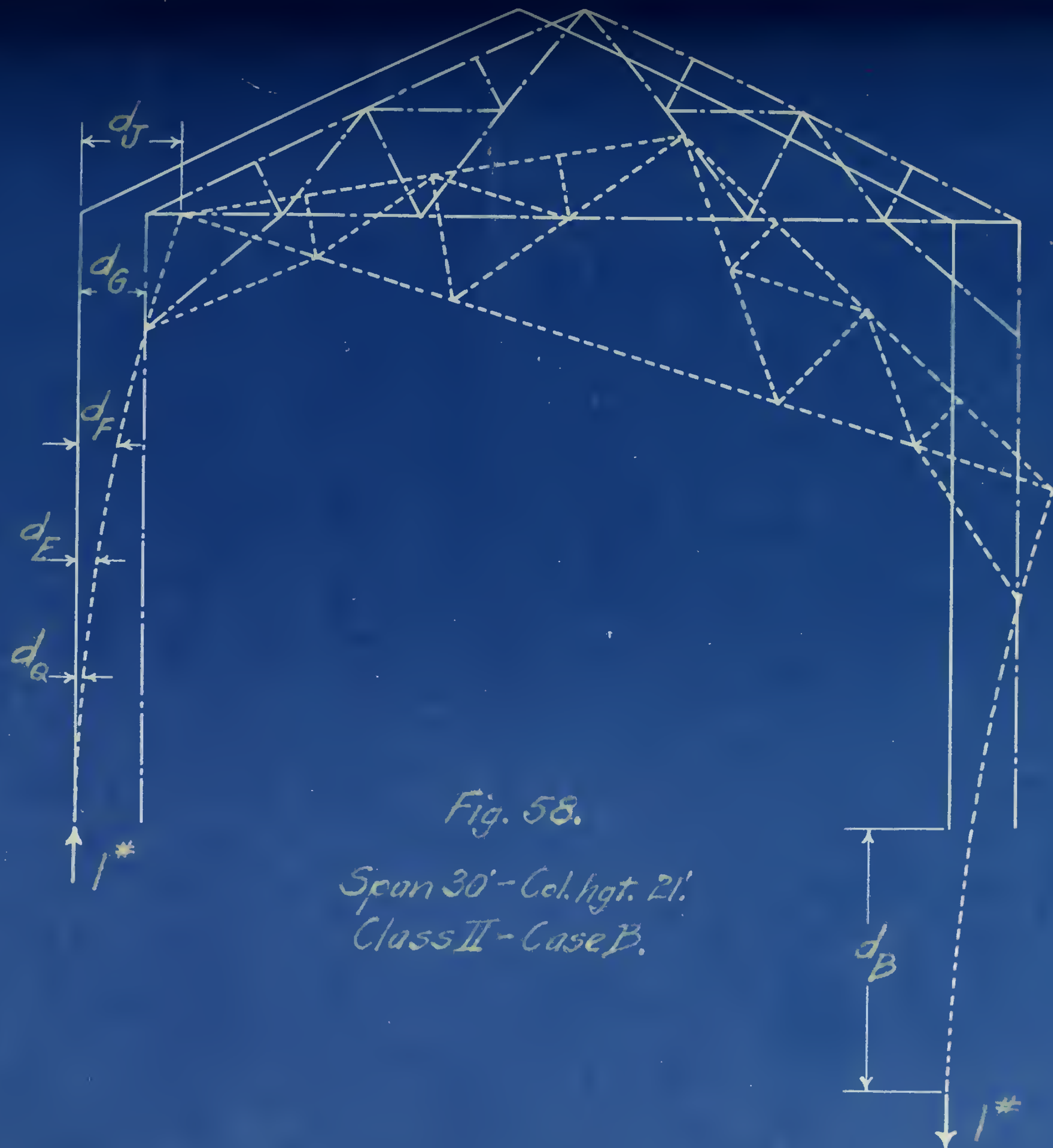


Fig. 58.

Span 30' - Col. hgt. 21'.
Class II - Case B.

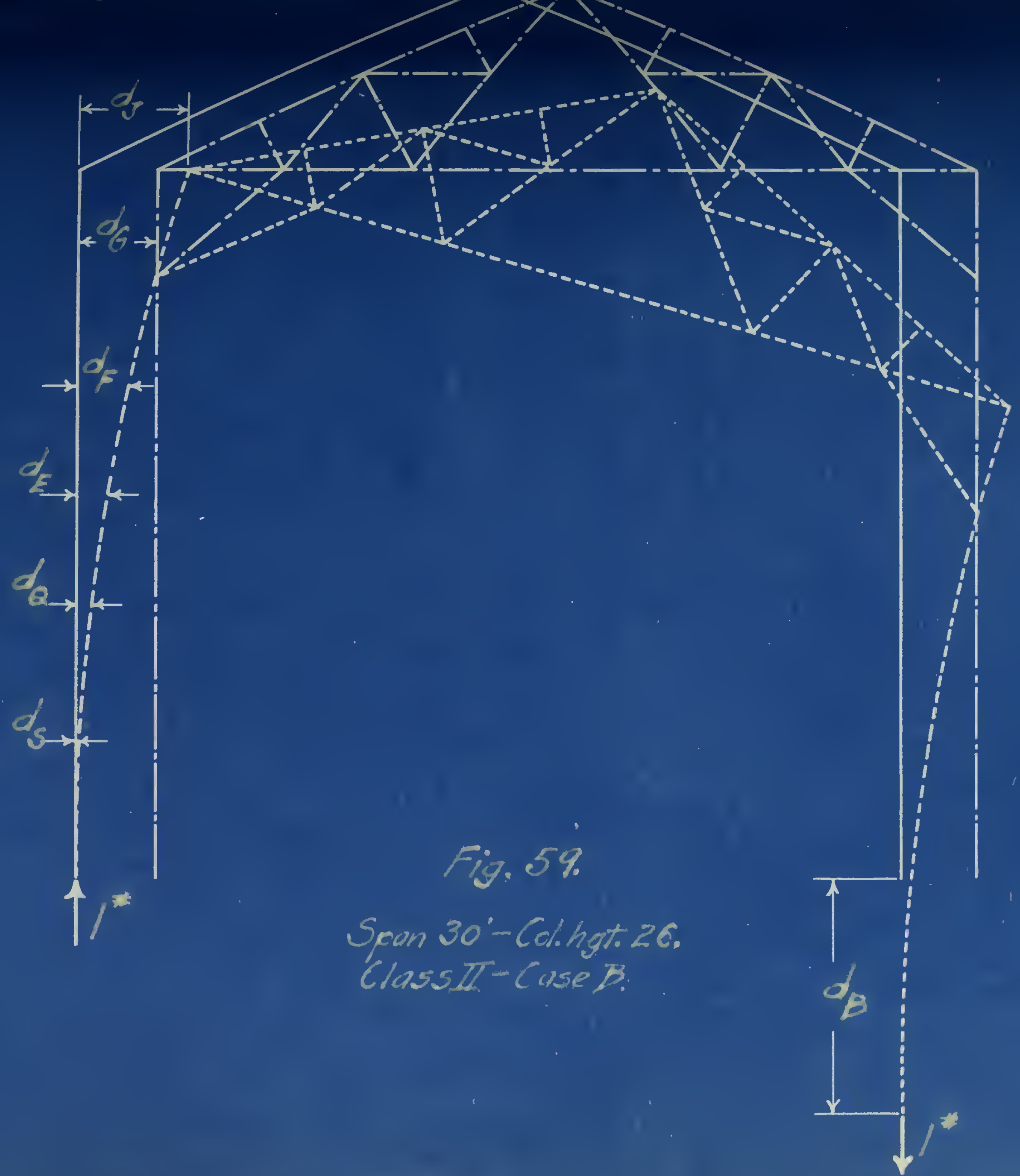


Fig. 59.

Span 30' - Col. hgt. 26.
Class II - Case B.

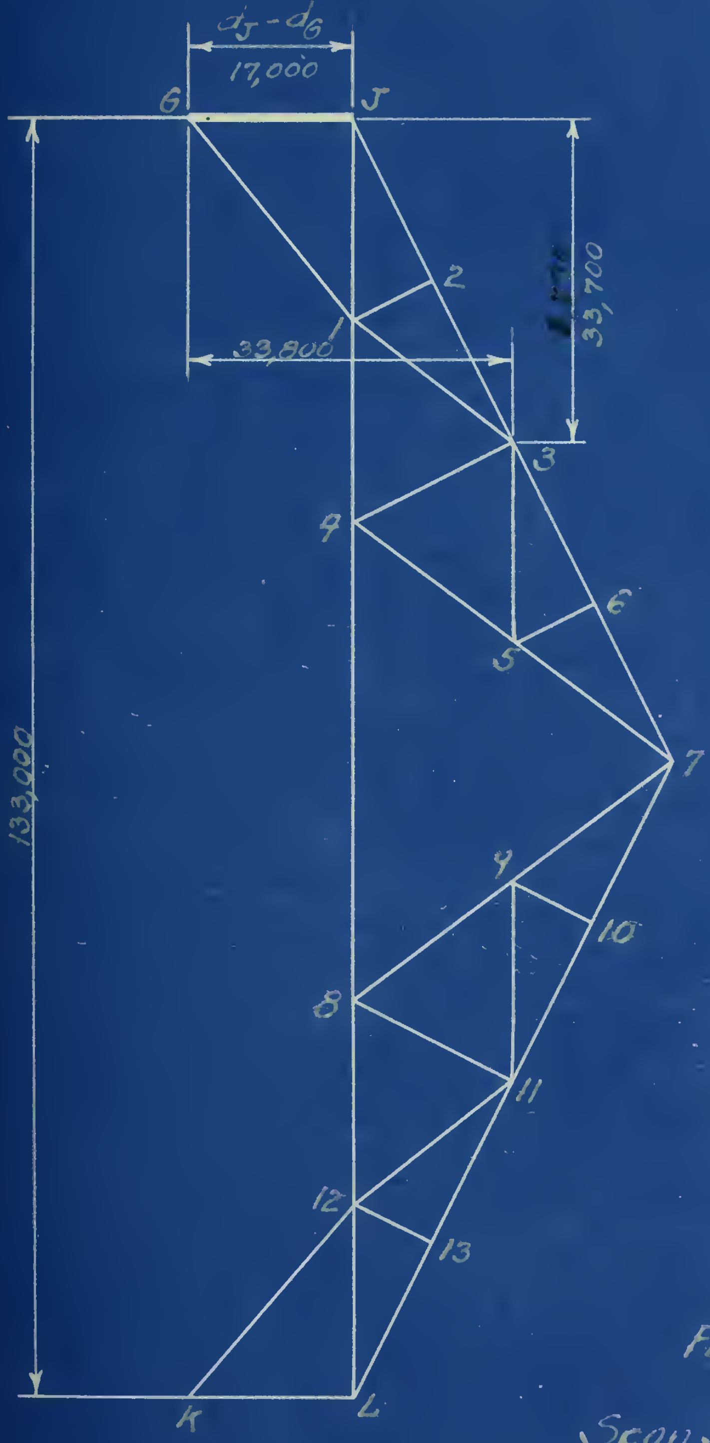


Fig. 61.

Span 30 - Col. hgt. 16'.
Class II - Case B.

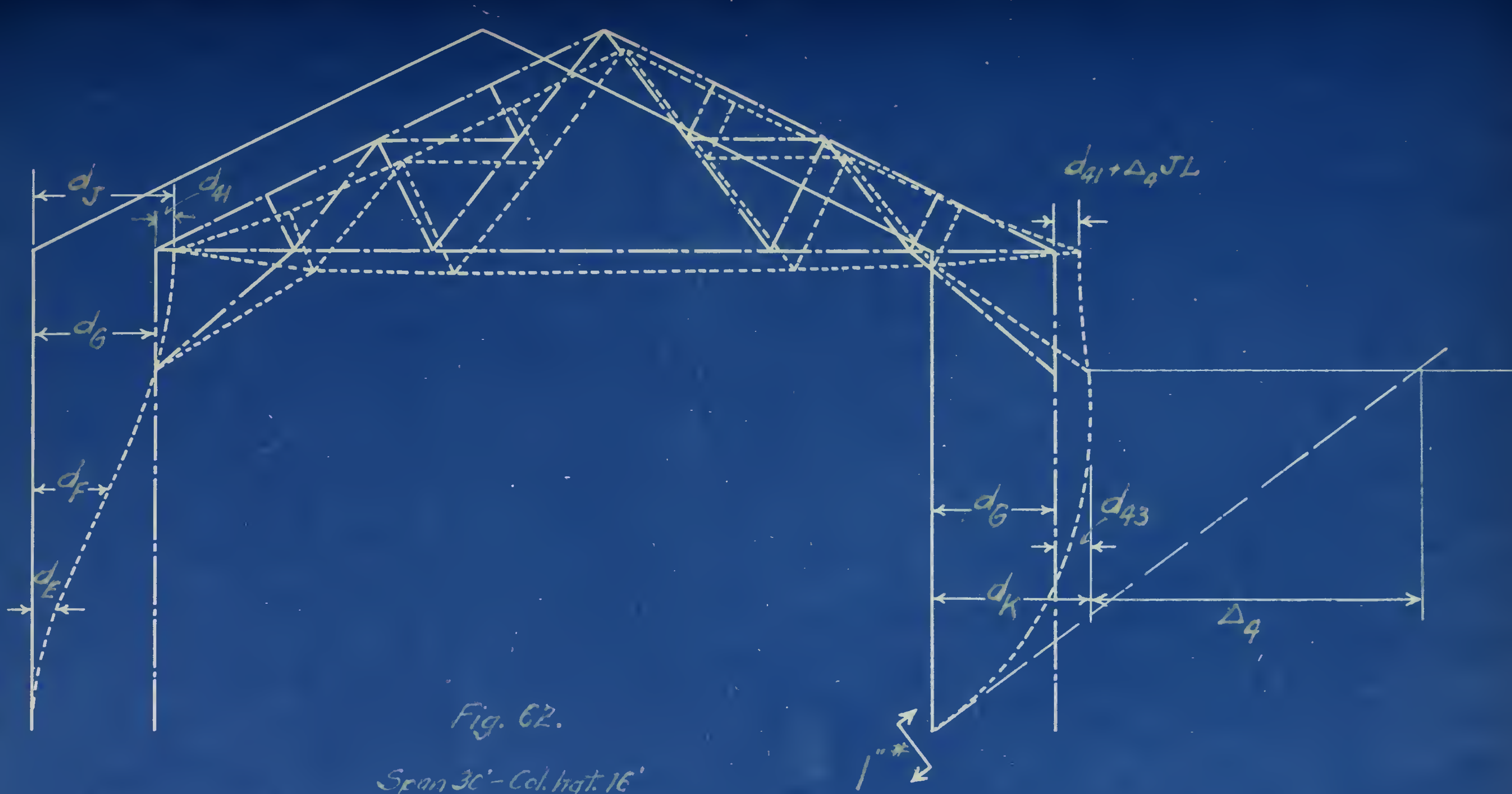


Fig. 62.

Span 30' - Col. hgt. 16'
Class II - Case C.

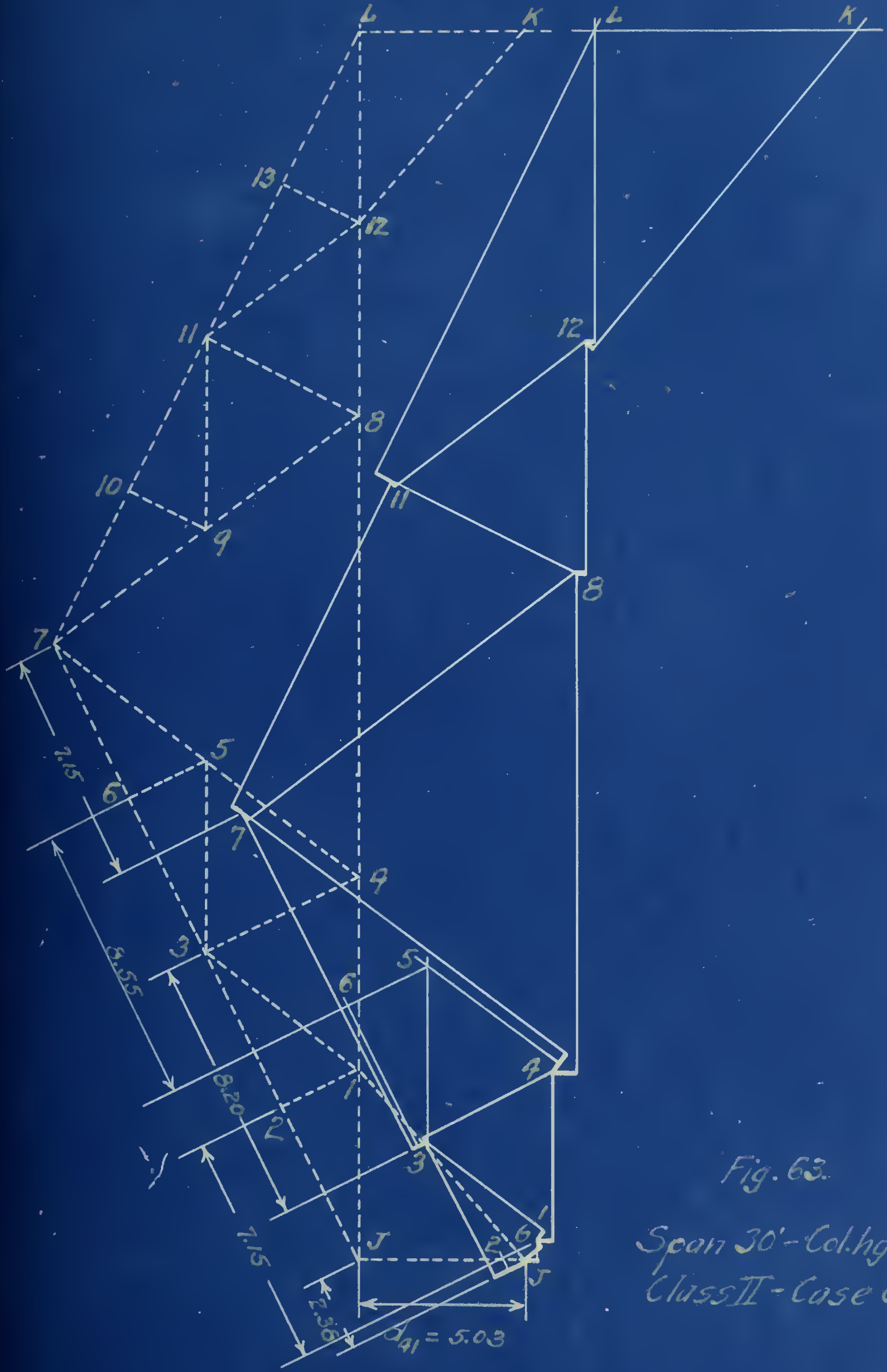


Fig. 63.

Span 30' - Col. hgt. 16'.
Class II - Case C.

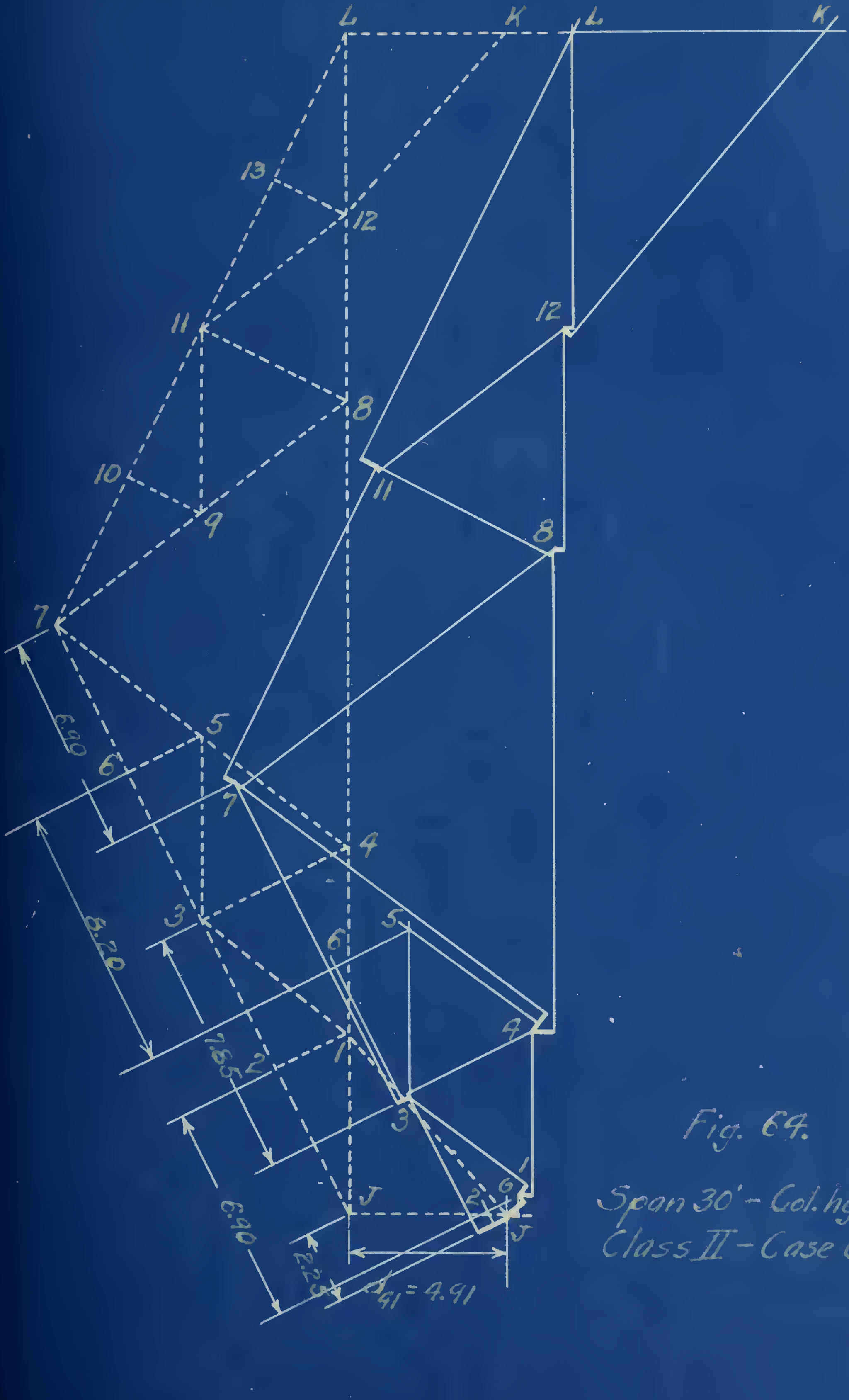


Fig. 64.

Span 30' - Col. hgt. 21'.
Class II - Case C.

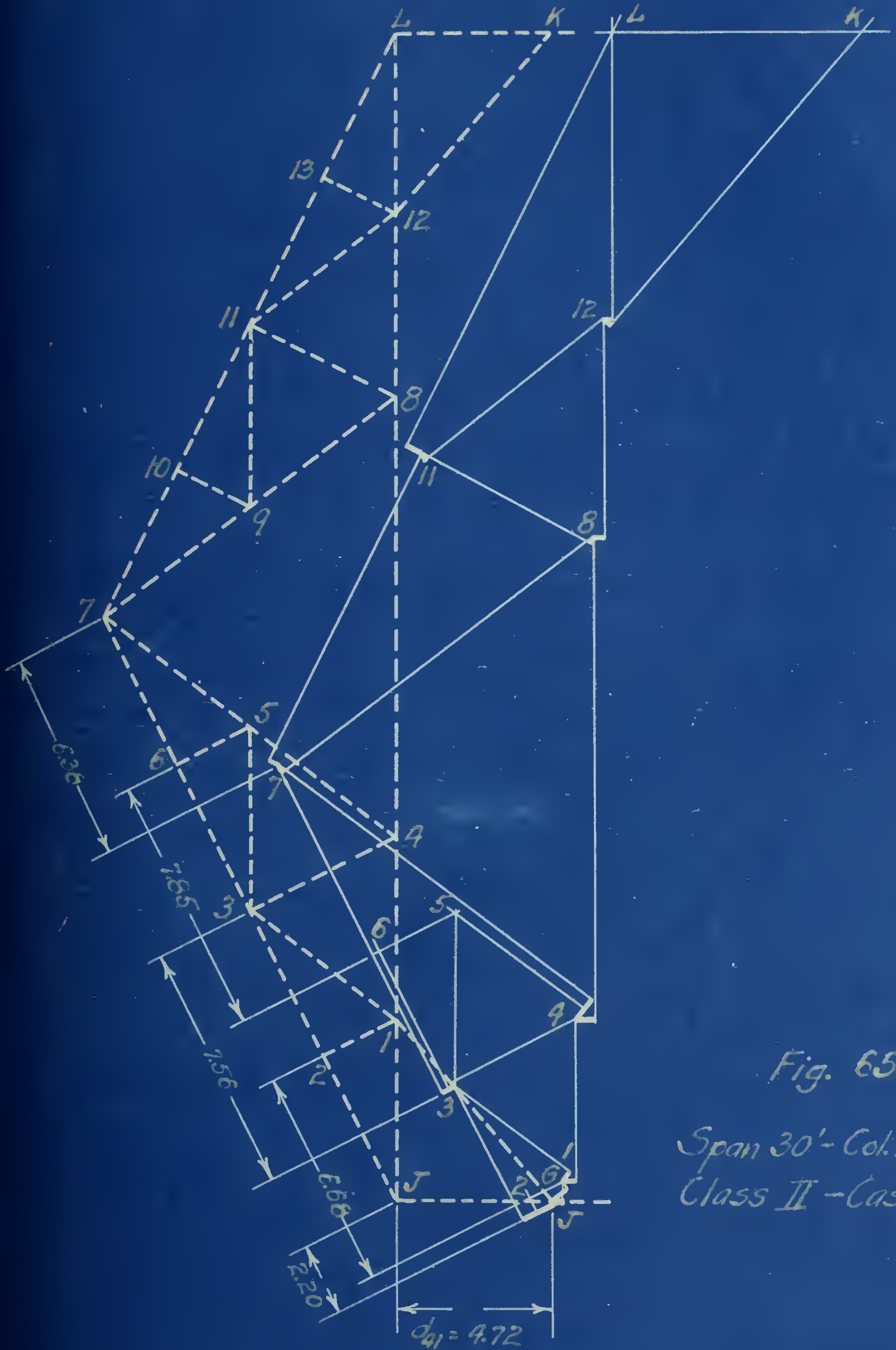


Fig. 65.

Span 30'- Col. hgt. 26'
Class II - Case C.

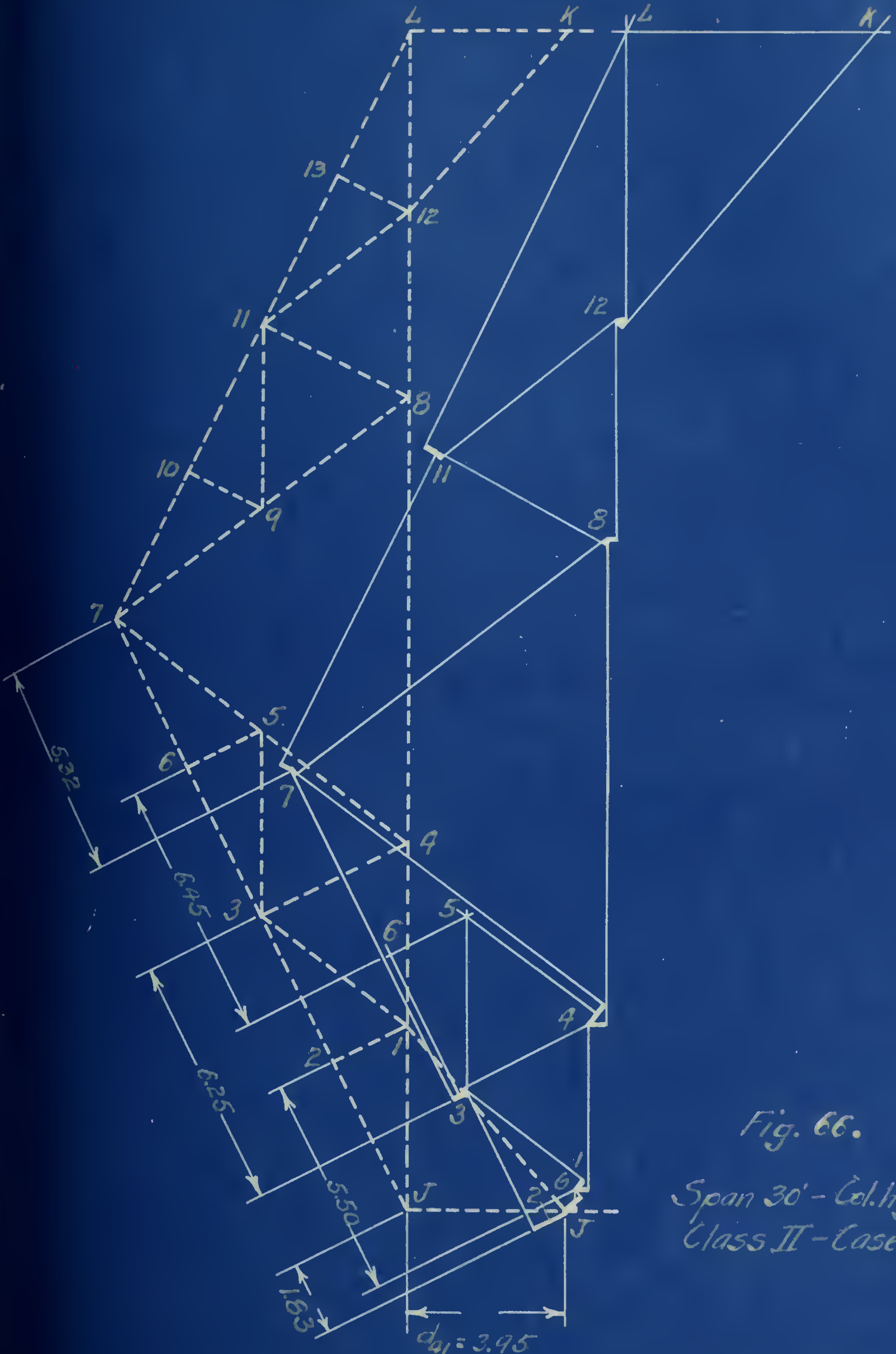


Fig. 66.

Span 30' - Col. hgt. 31.'
Class II - Case C.

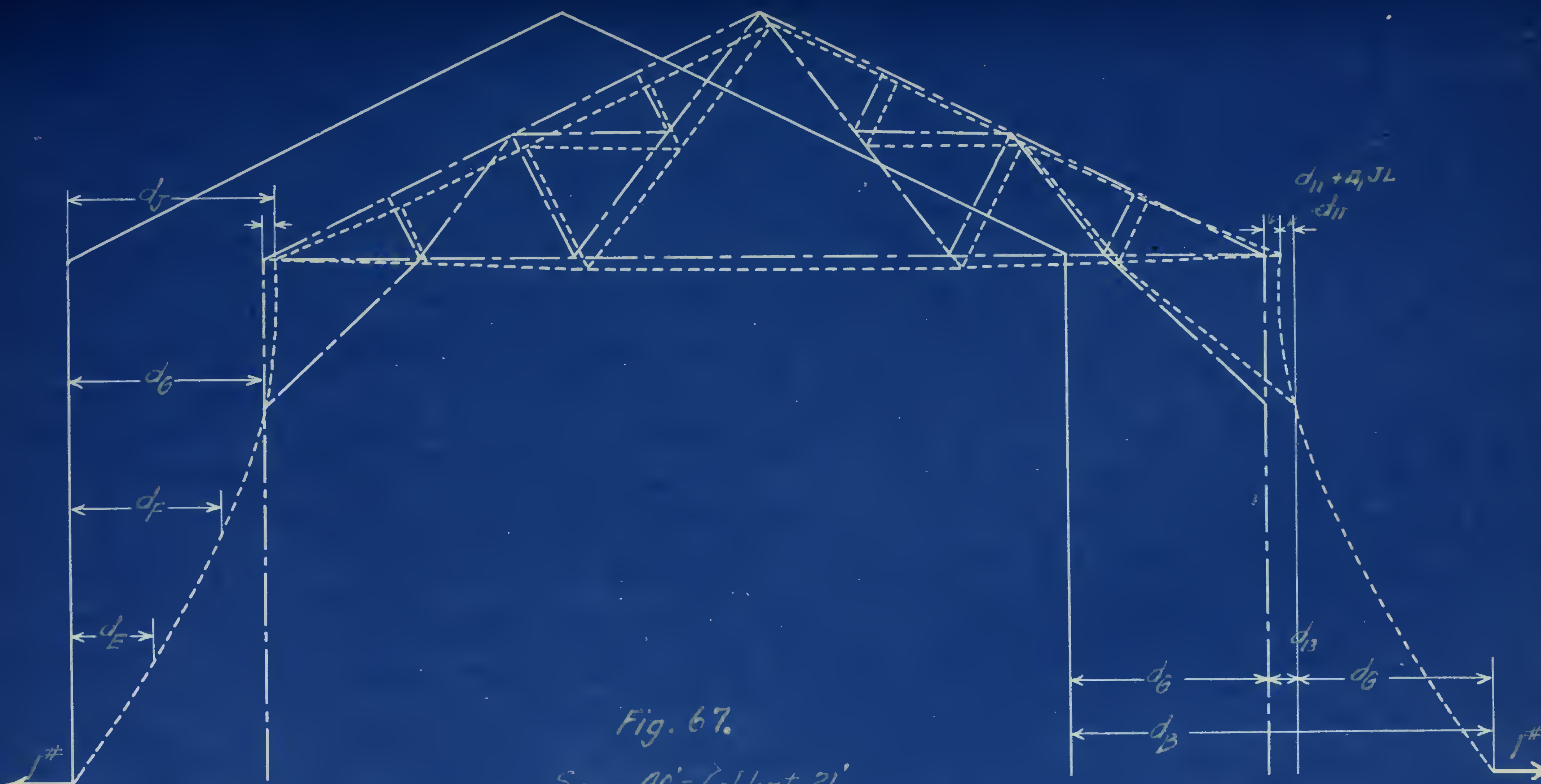


Fig. 67.

Span 40'-Col. hgt. 21'.
Class I-Case A.

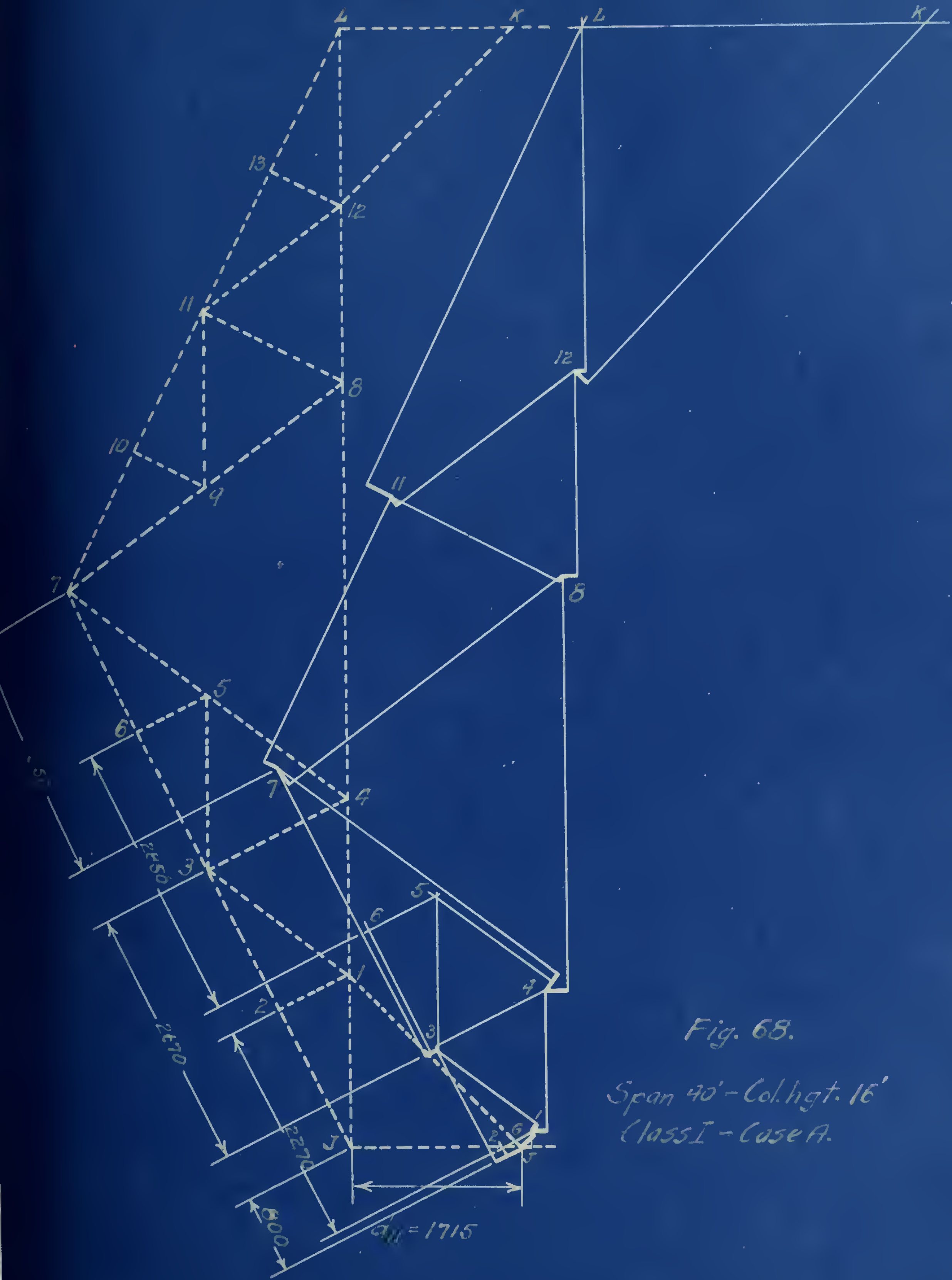


Fig. 68.

Span 40' - Col. hgt. 16'
Class I - Case A.

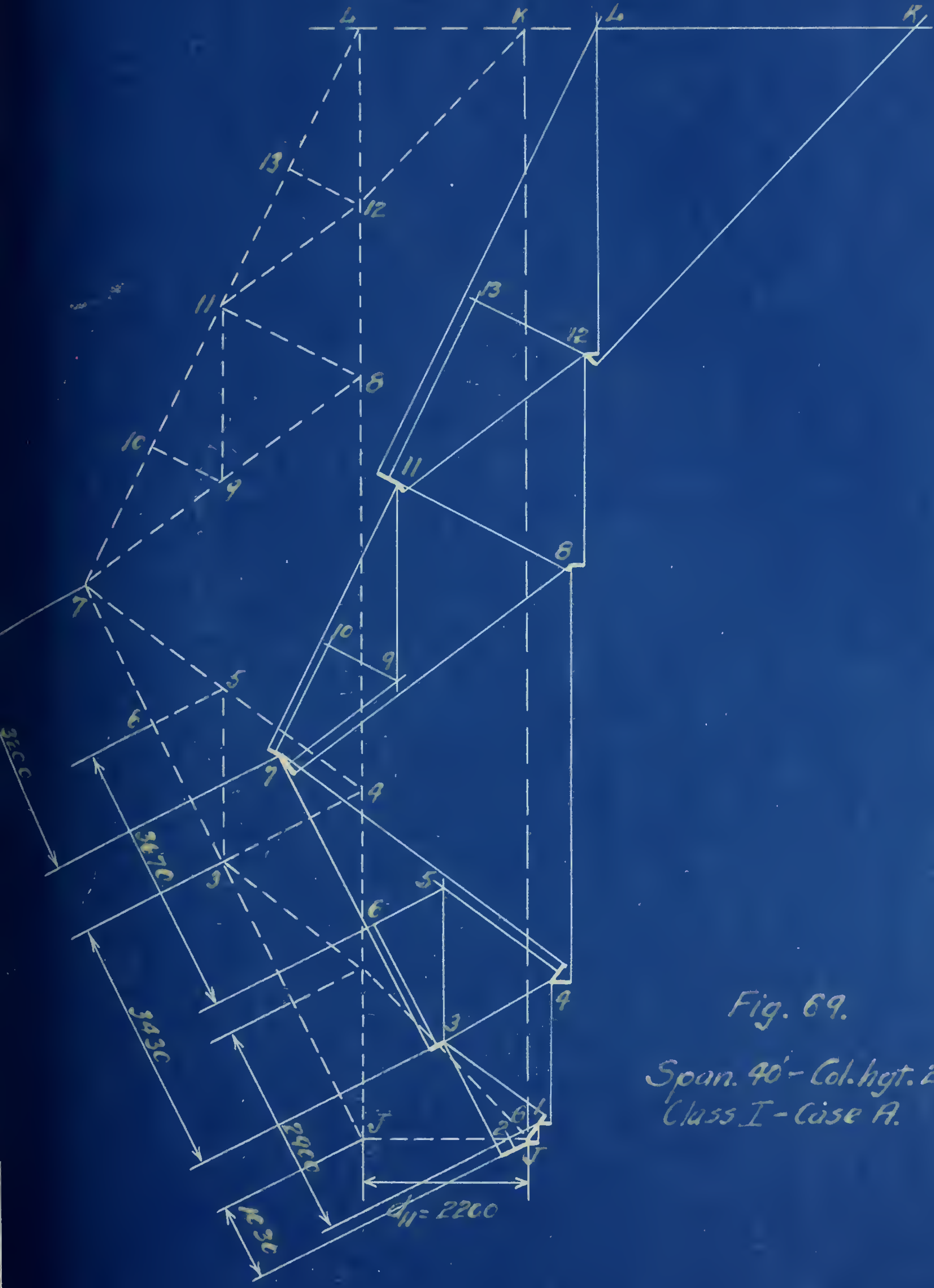


Fig. 69.

Span. 40' - Col. hgt. 21'.
Class I - Case A.

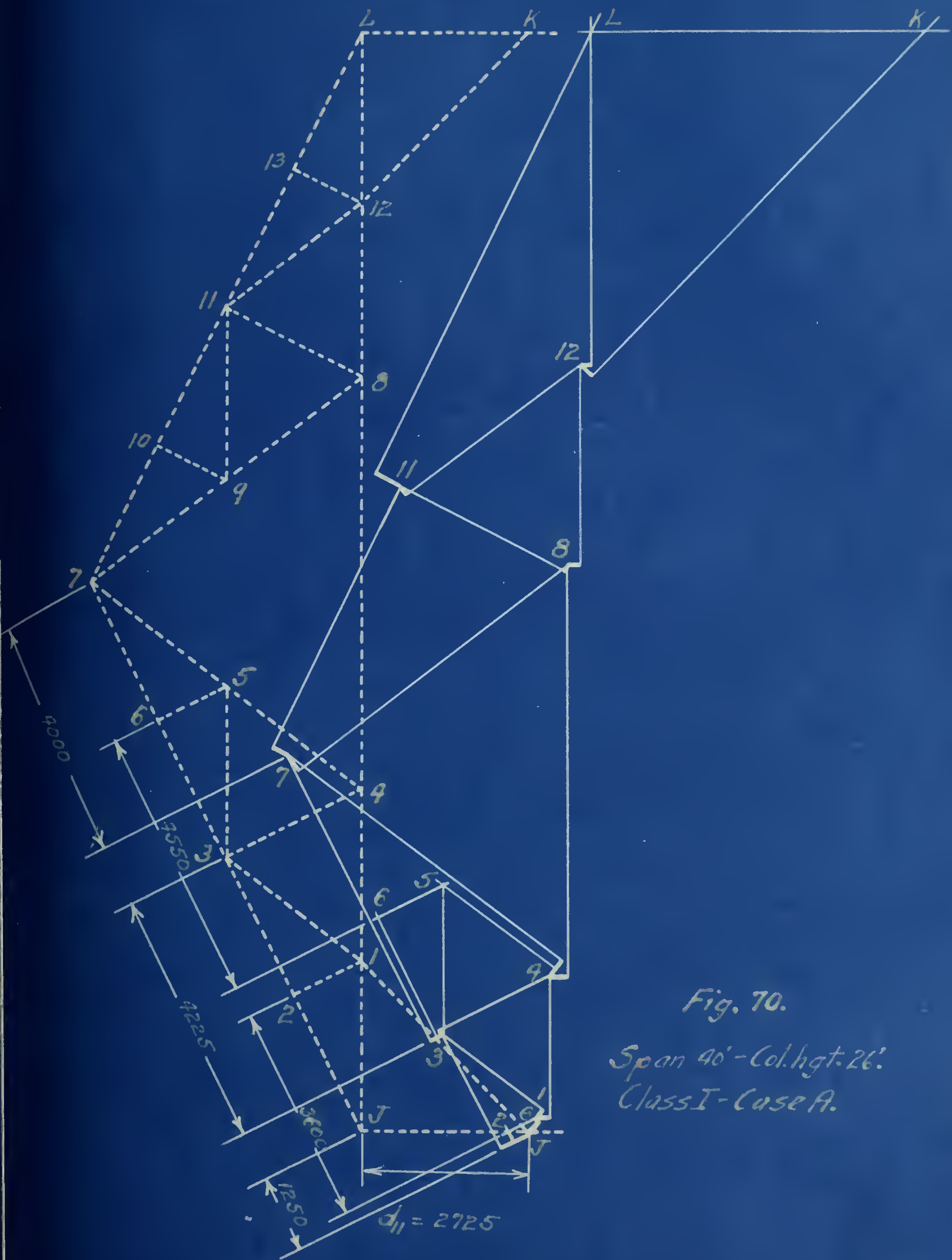


Fig. 70.

Span 40' - Col. hgt. 26'.
Class I - Case A.

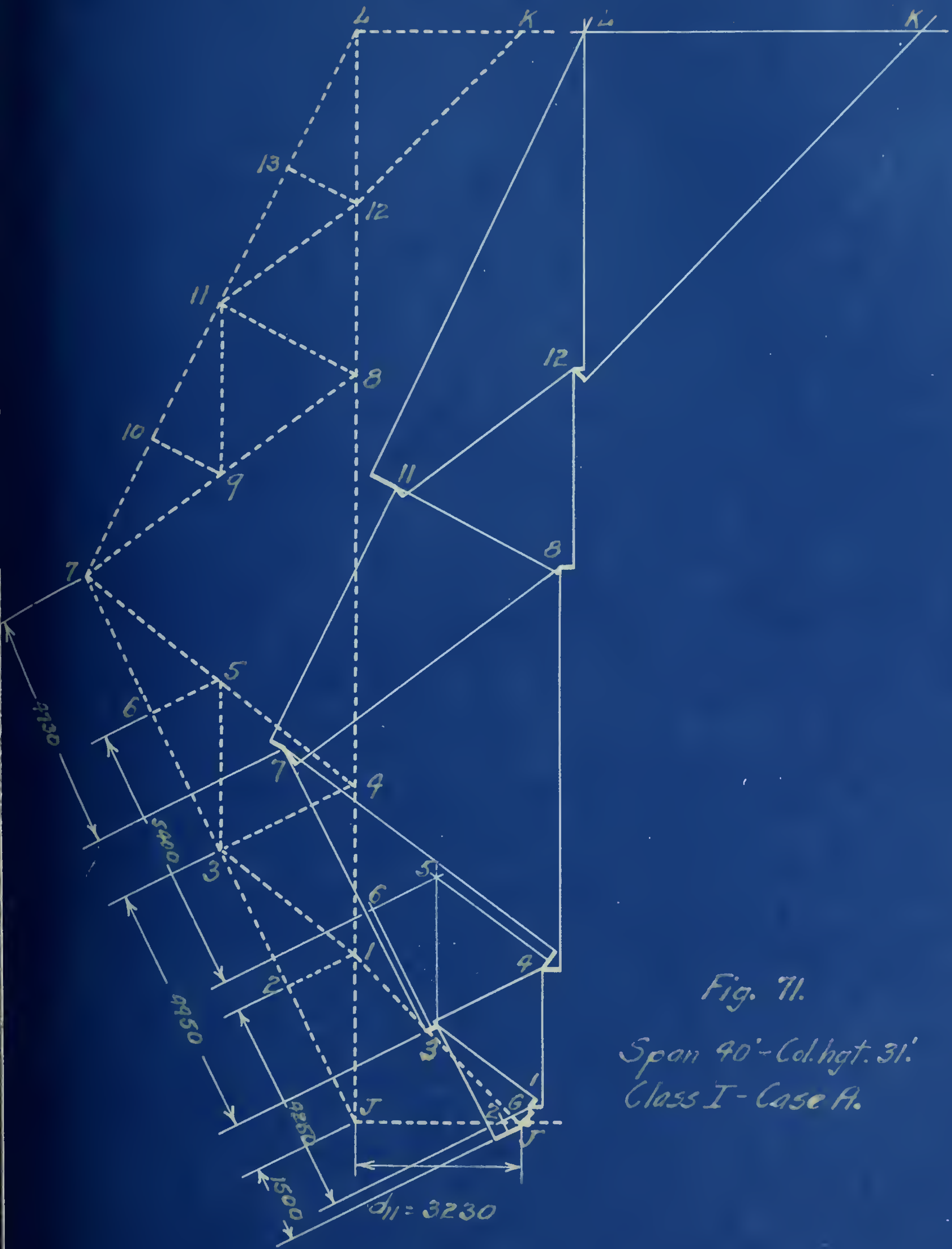


Fig. 71.

Span 40'-Col. hgt. 31'.
Class I-Case A.

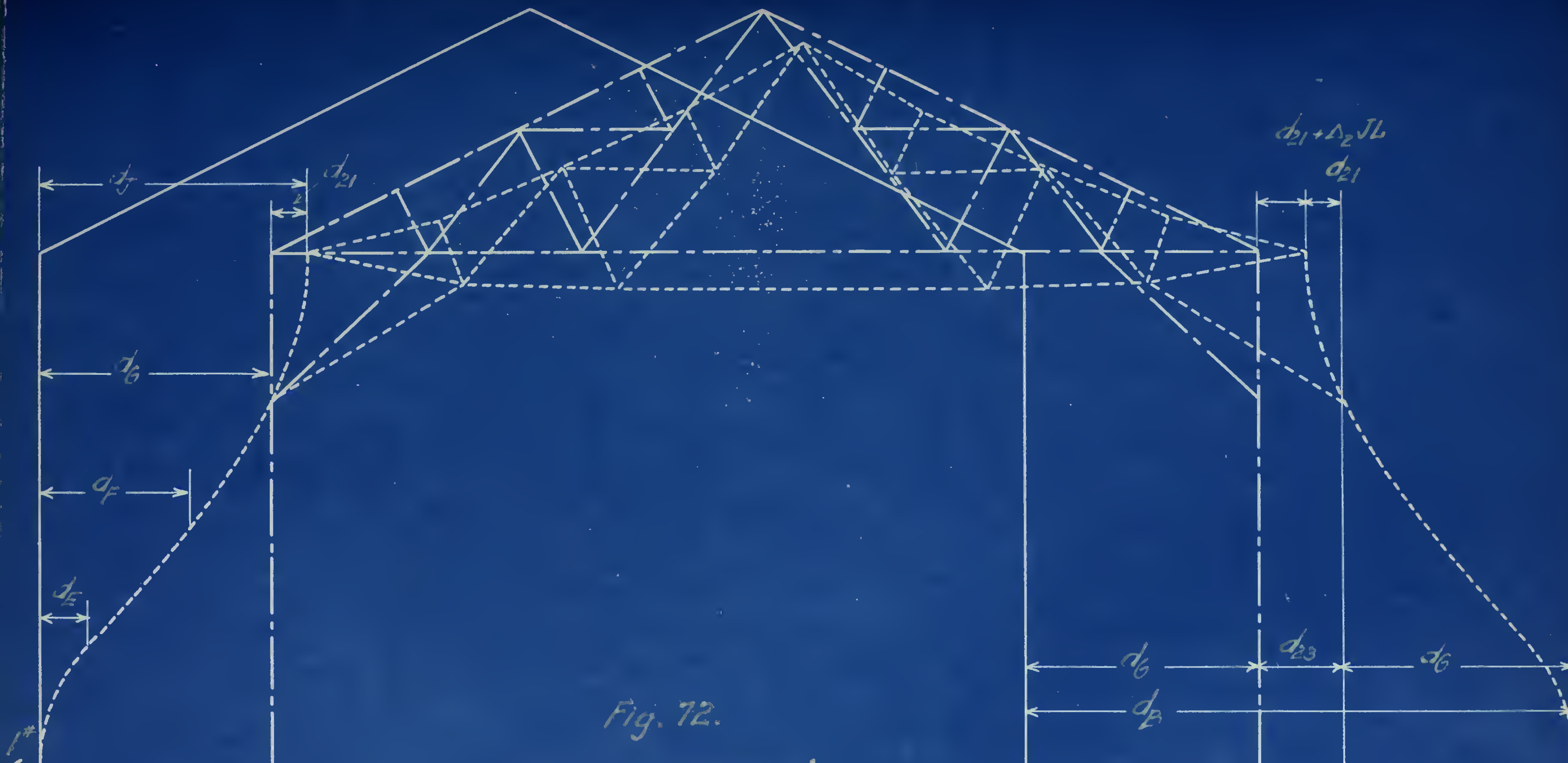


Fig. 72.

Span 40' - Col. hgt. 21'
Class II - Case A.

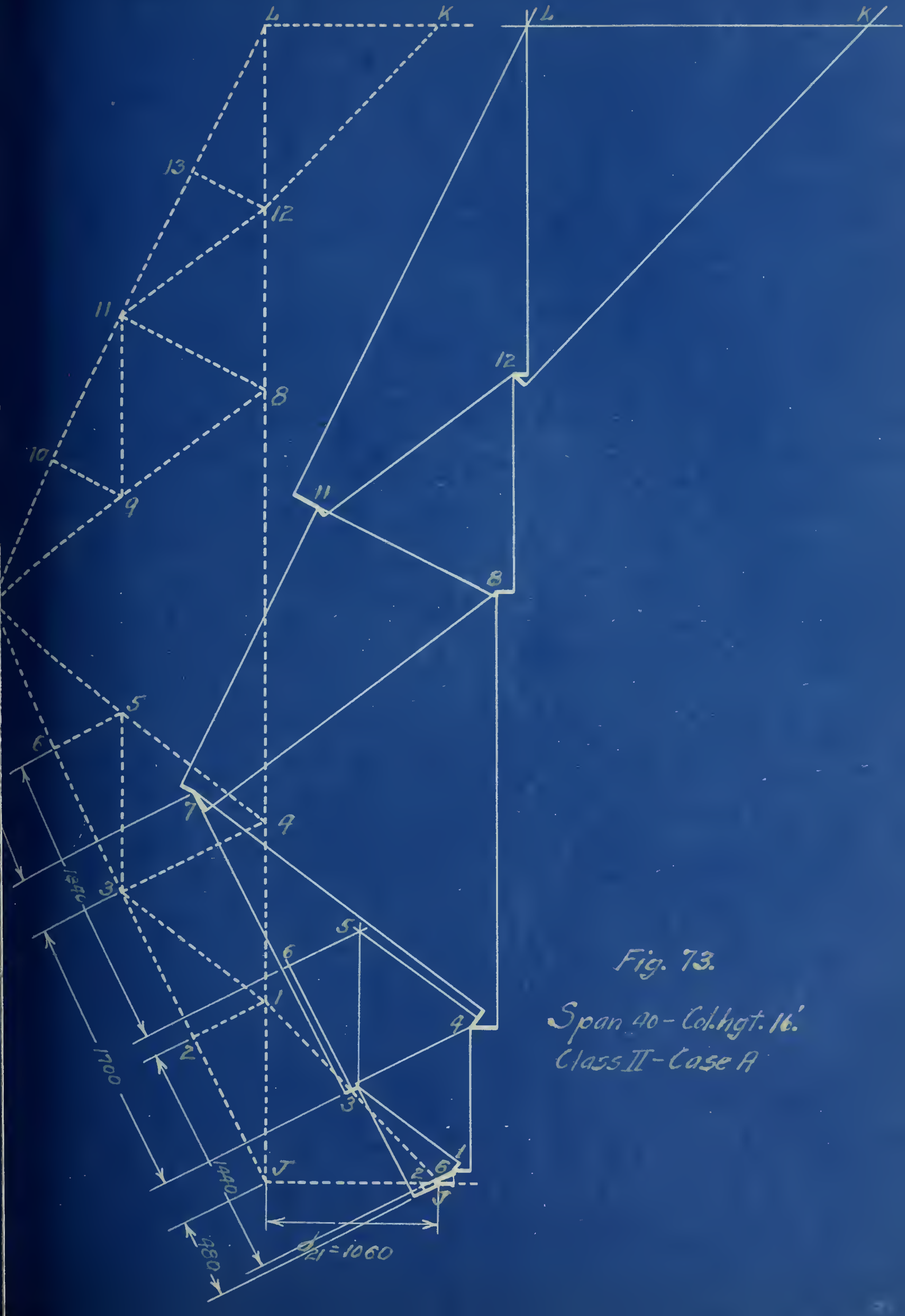


Fig. 73.

Span 40 - Col. hgt. 16'
Class II - Case A

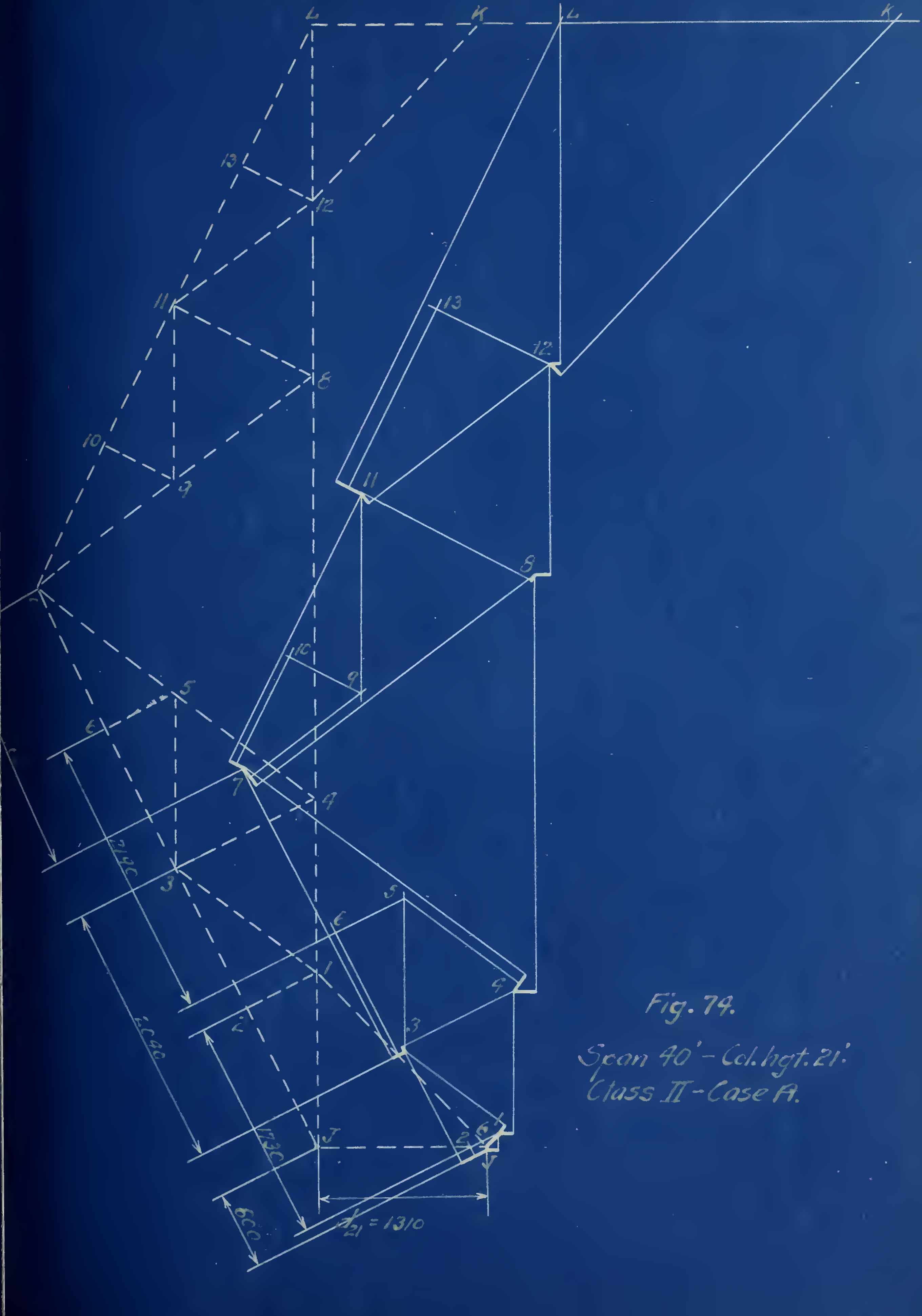


Fig. 74.

Span 40' - Col. hgt. 21'.
Class II - Case A.

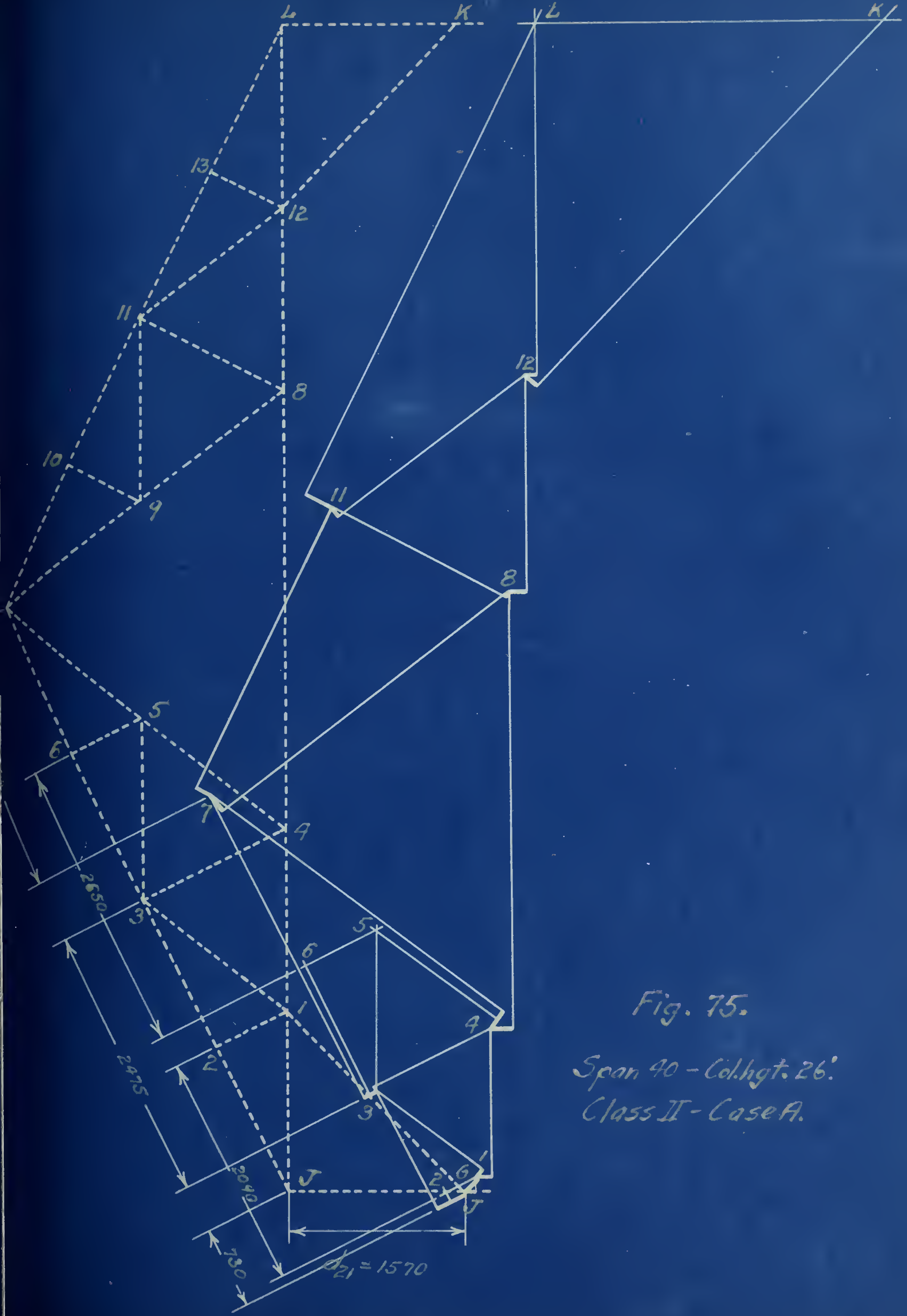


Fig. 75.

Span 40 - Col. hgt. 26'.
Class II - Case A.

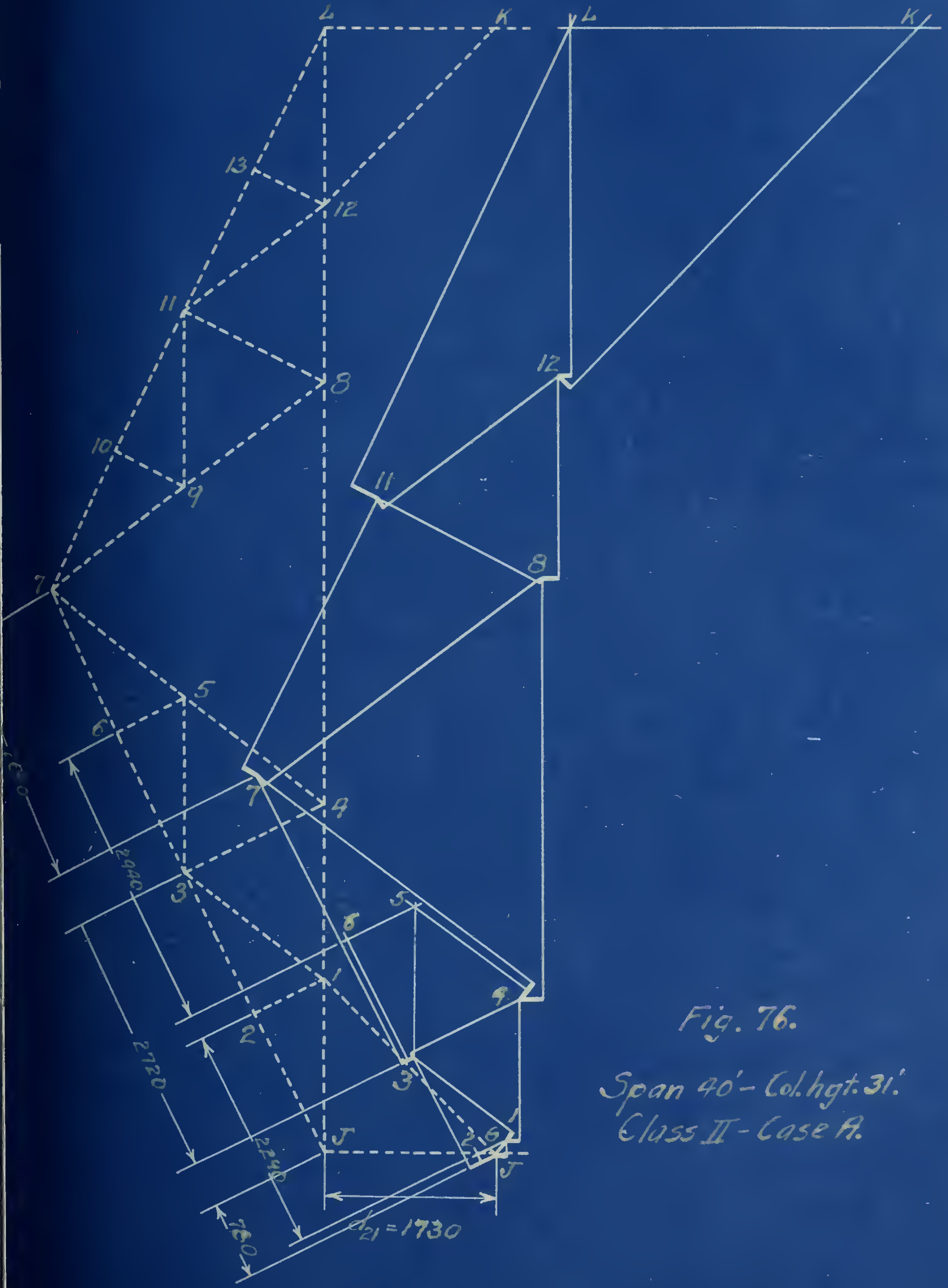


Fig. 76.

Span 40' - Col. hgt. 31'.
Class II - Case A.

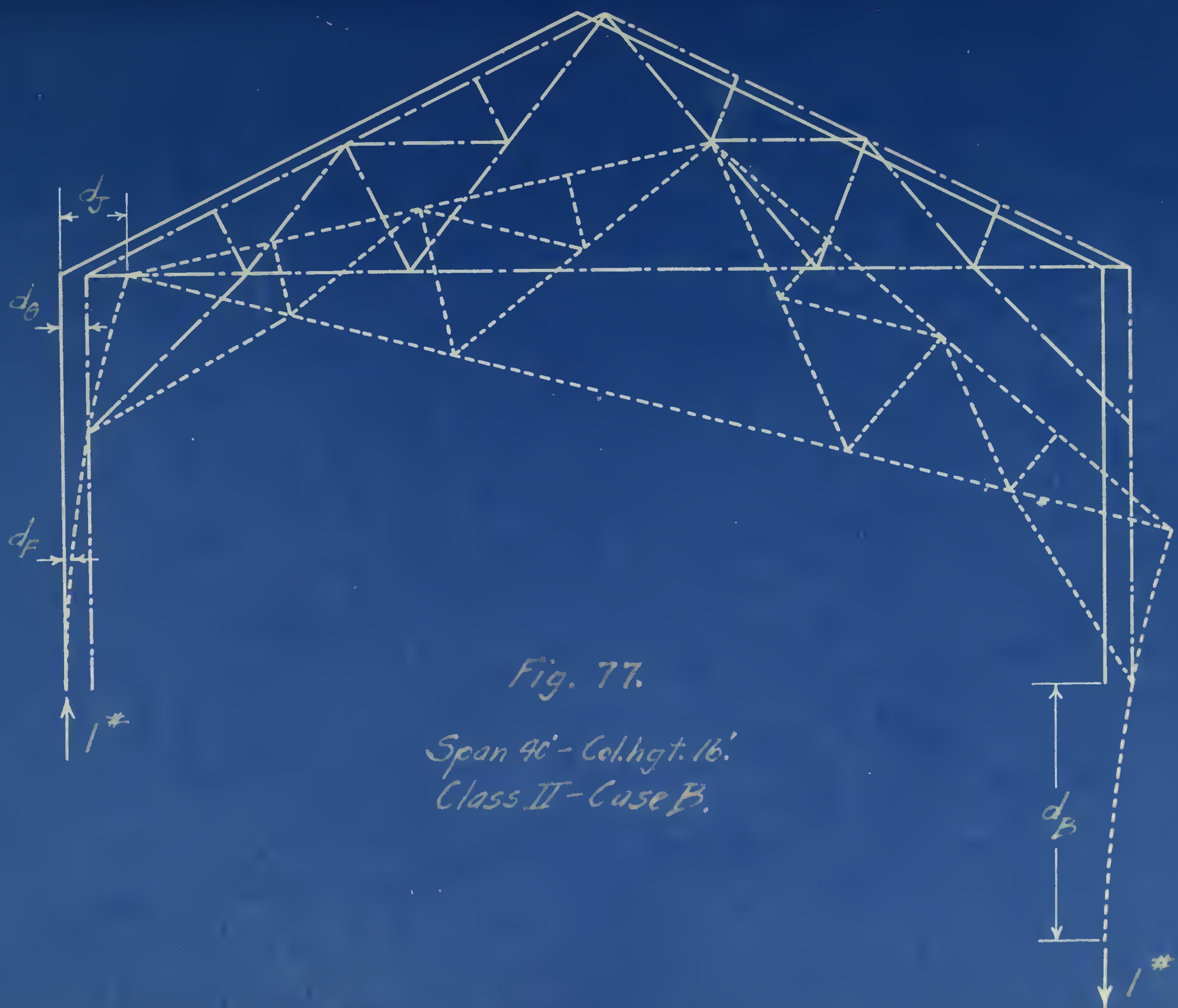


Fig. 77.

Span 40' - Col. hgt. 16'.
Class II - Case B.

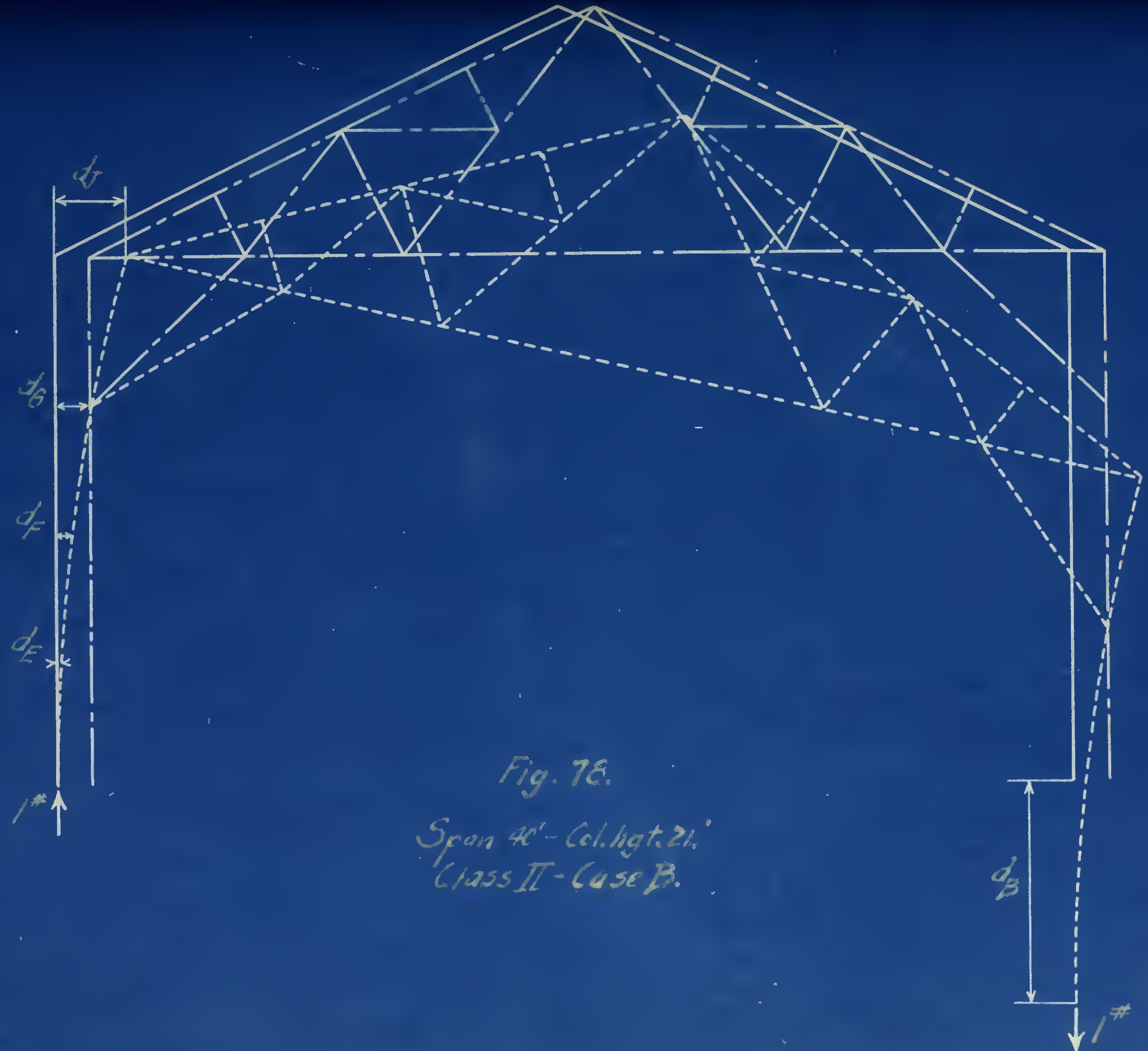


Fig. 78.

Span 40' - Col. hgt. 21.'
Class II - Case B.

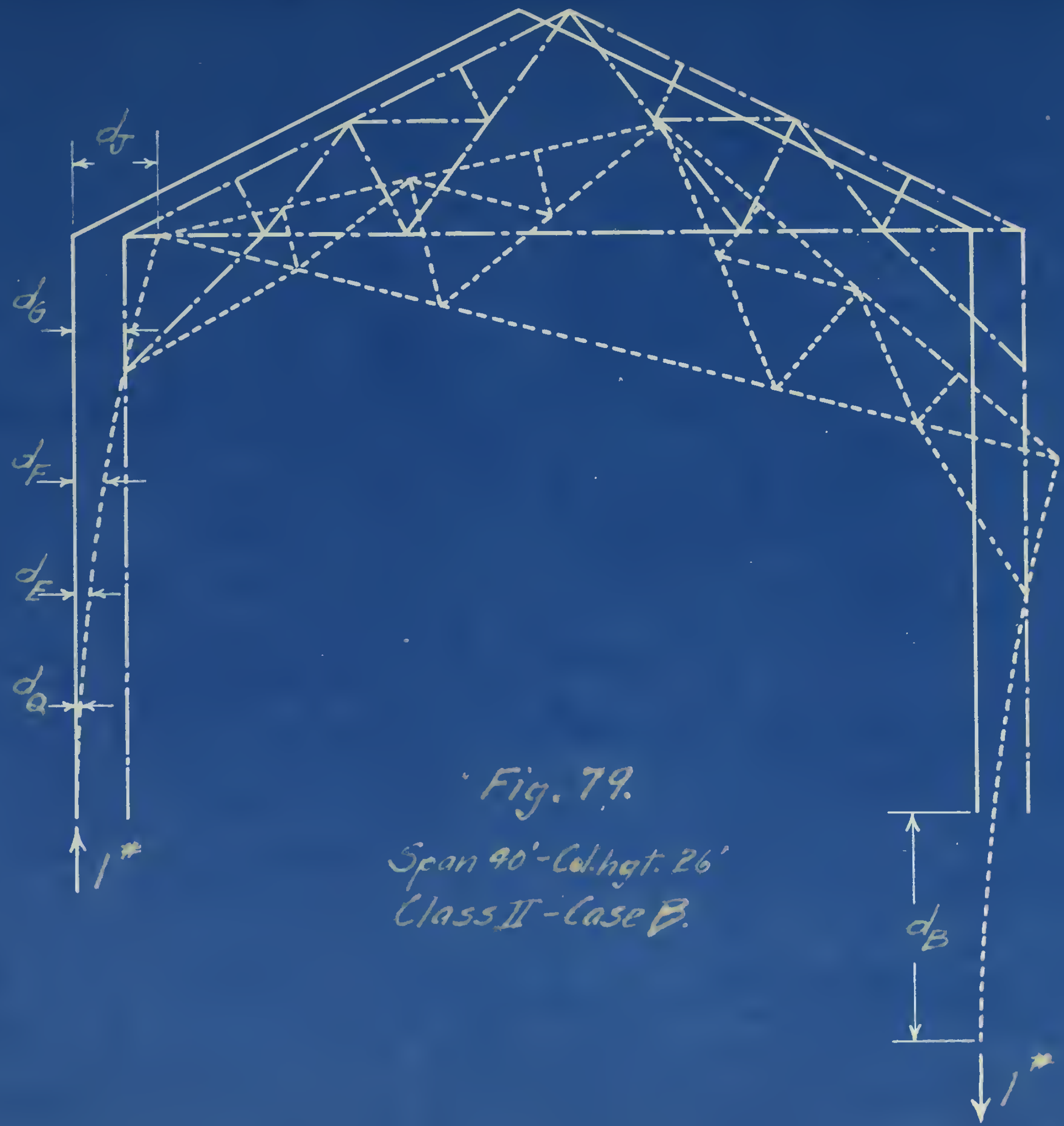


Fig. 79.

Span 90'-Col. hgt. 26'
Class II-Case B.

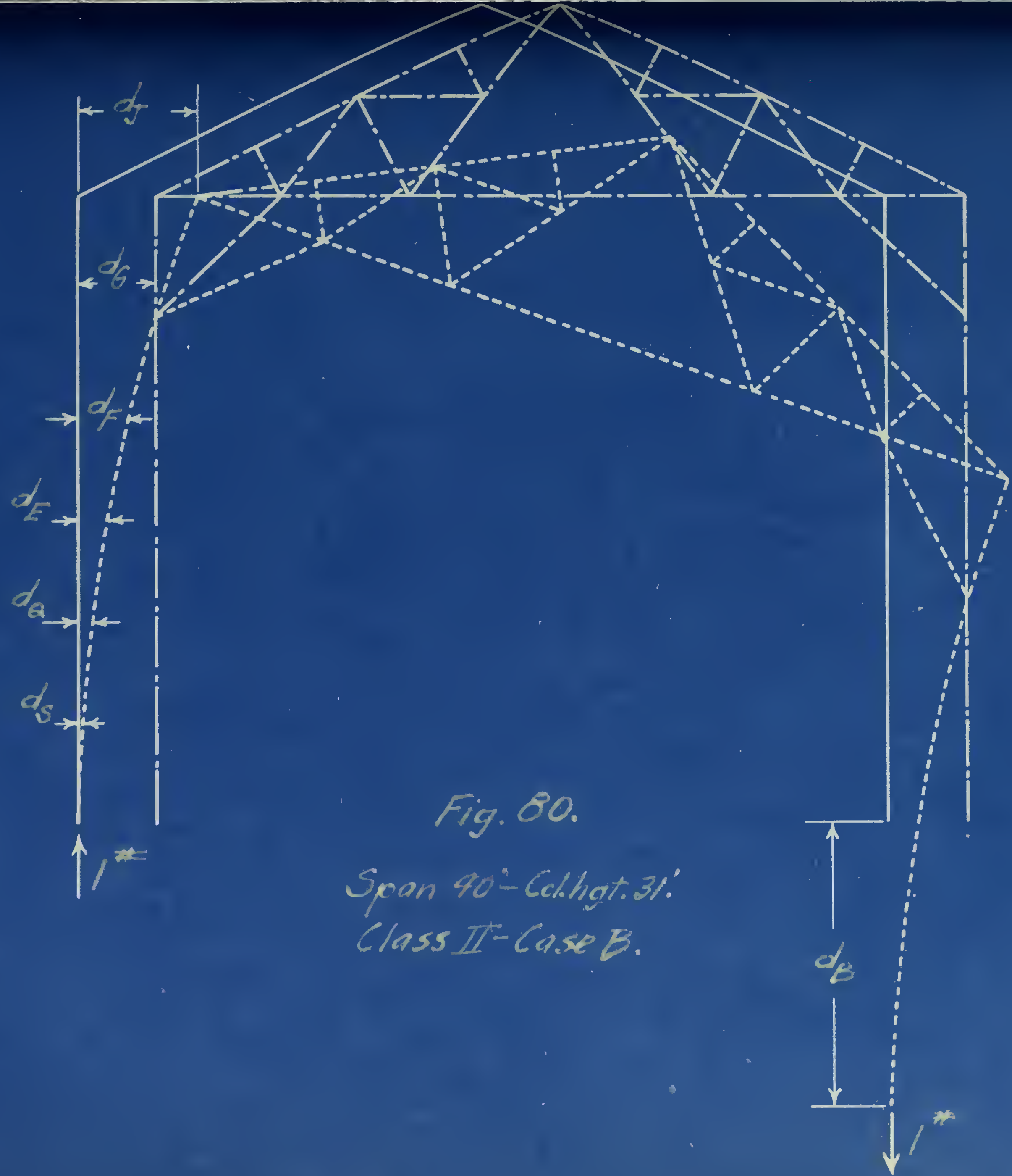


Fig. 80.

Span 40' - Col. hgt. 31'.
Class II - Case B.

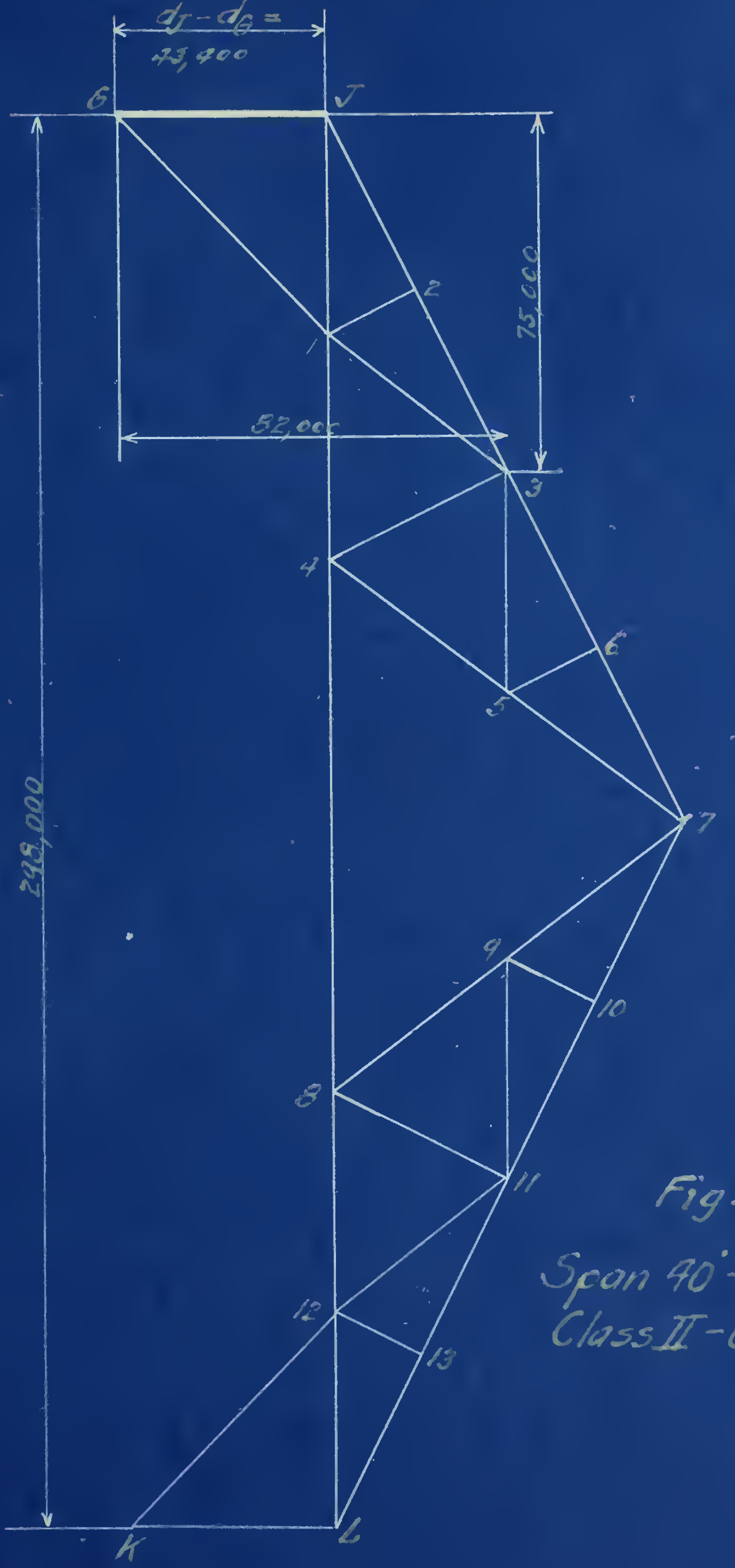


Fig. 81.

Span 40' - Col. hgt. 21.
Class II - Case B.

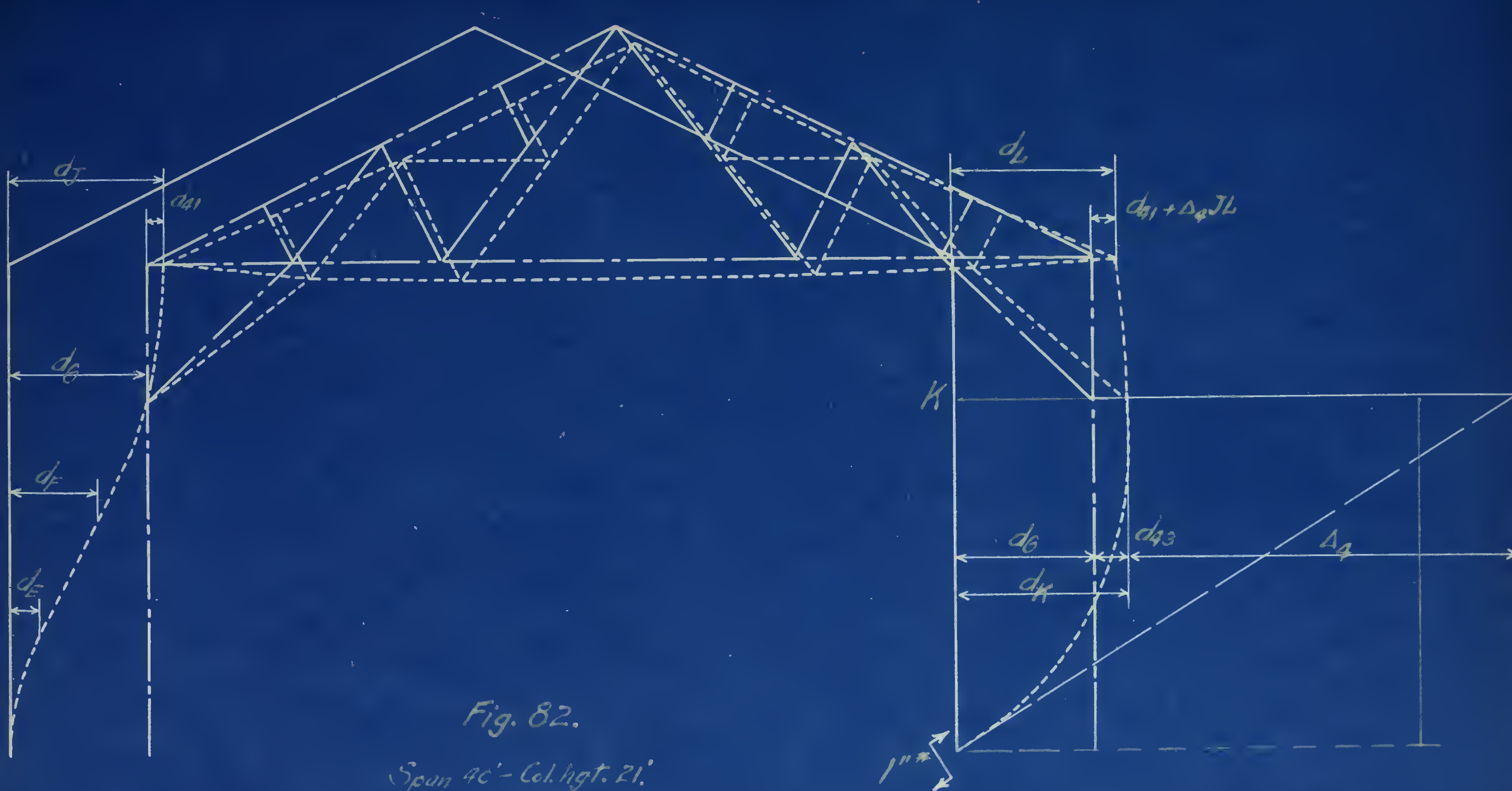


Fig. 82.

Span 40' - Col. hgt. 21'.
Class II - Case C.

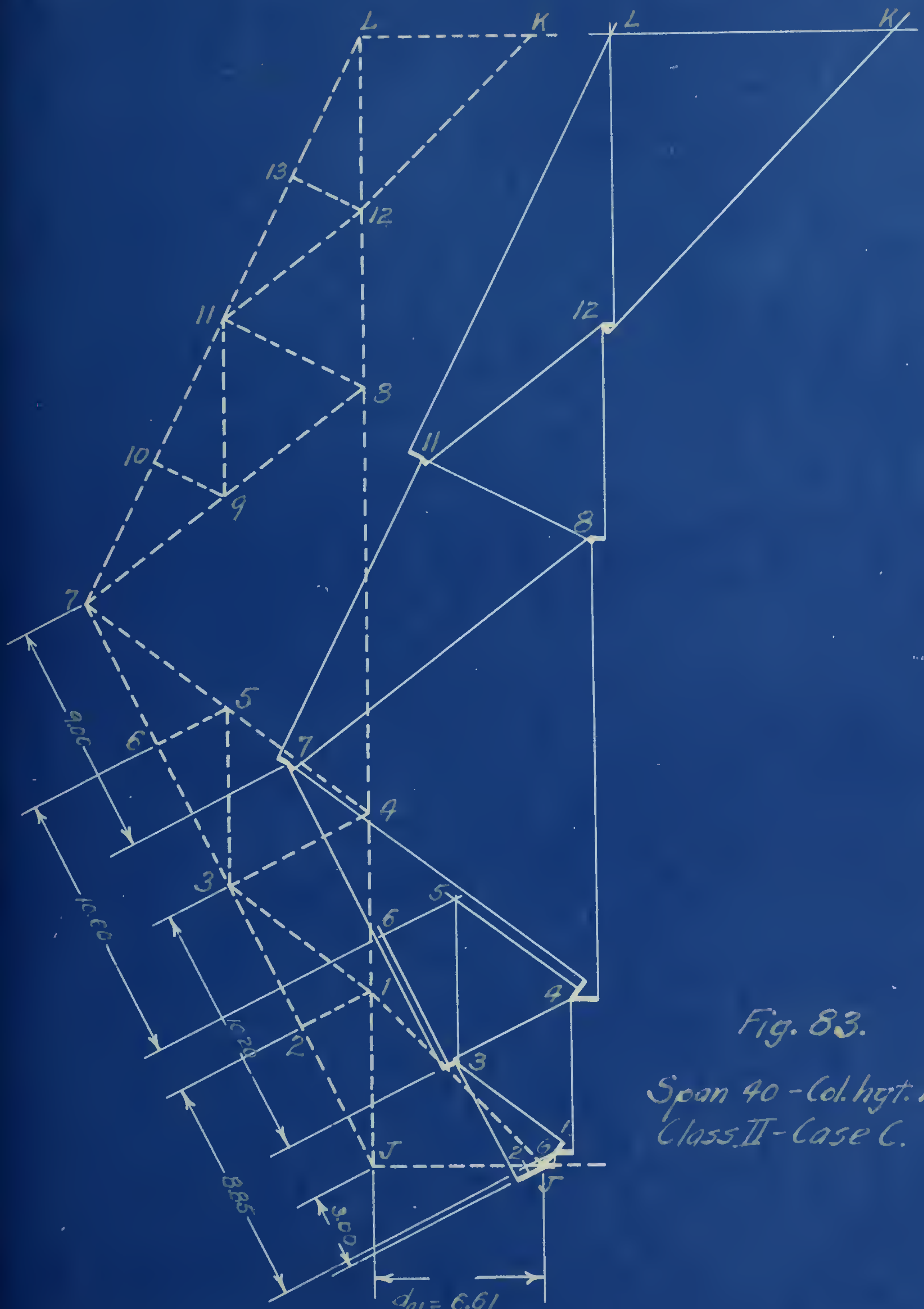


Fig. 83.

Span 40 - Col. hgt. 16'
Class II - Case C.

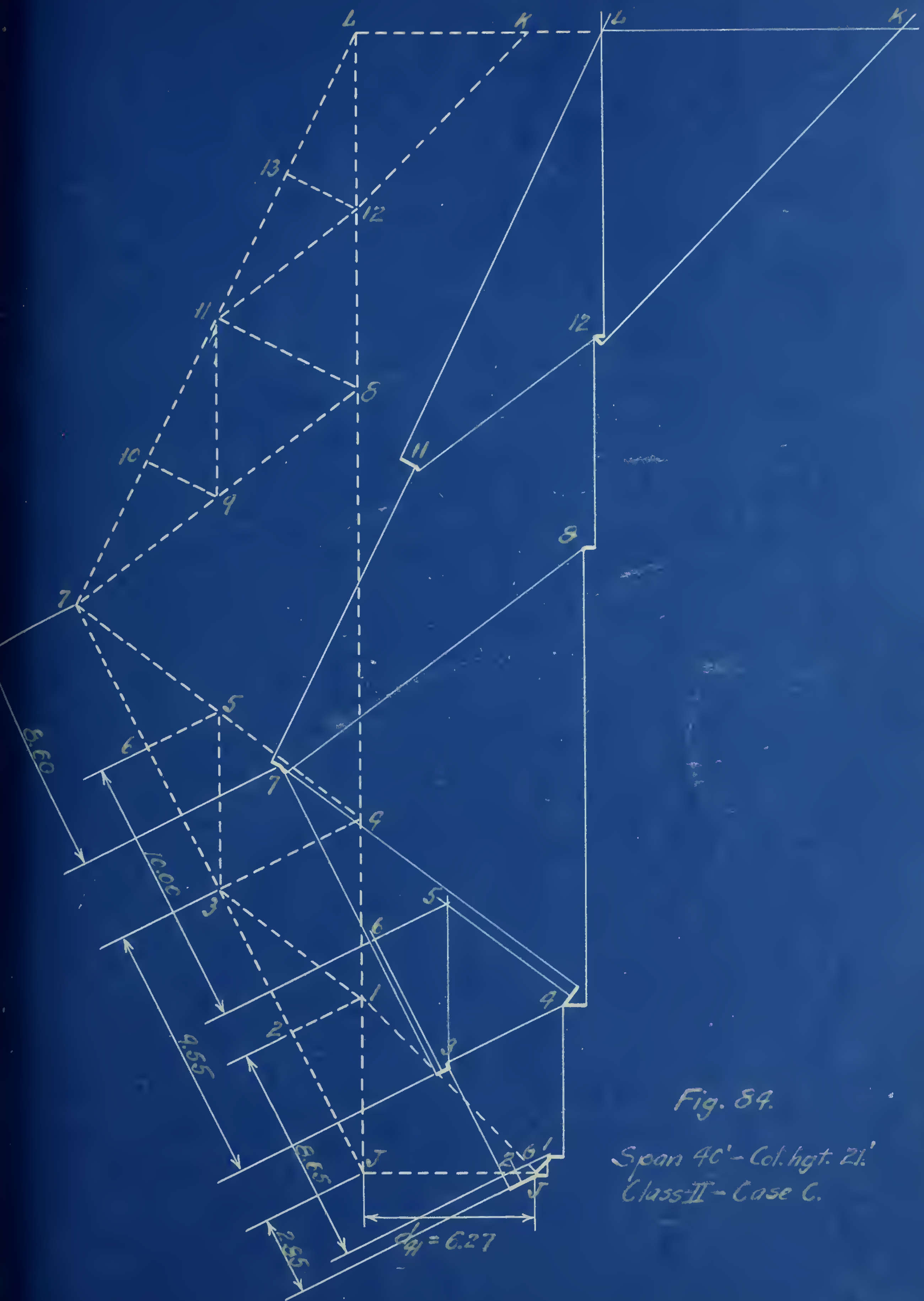


Fig. 84.

Span 40' – Col. hgt. 21'
Class II – Case C.

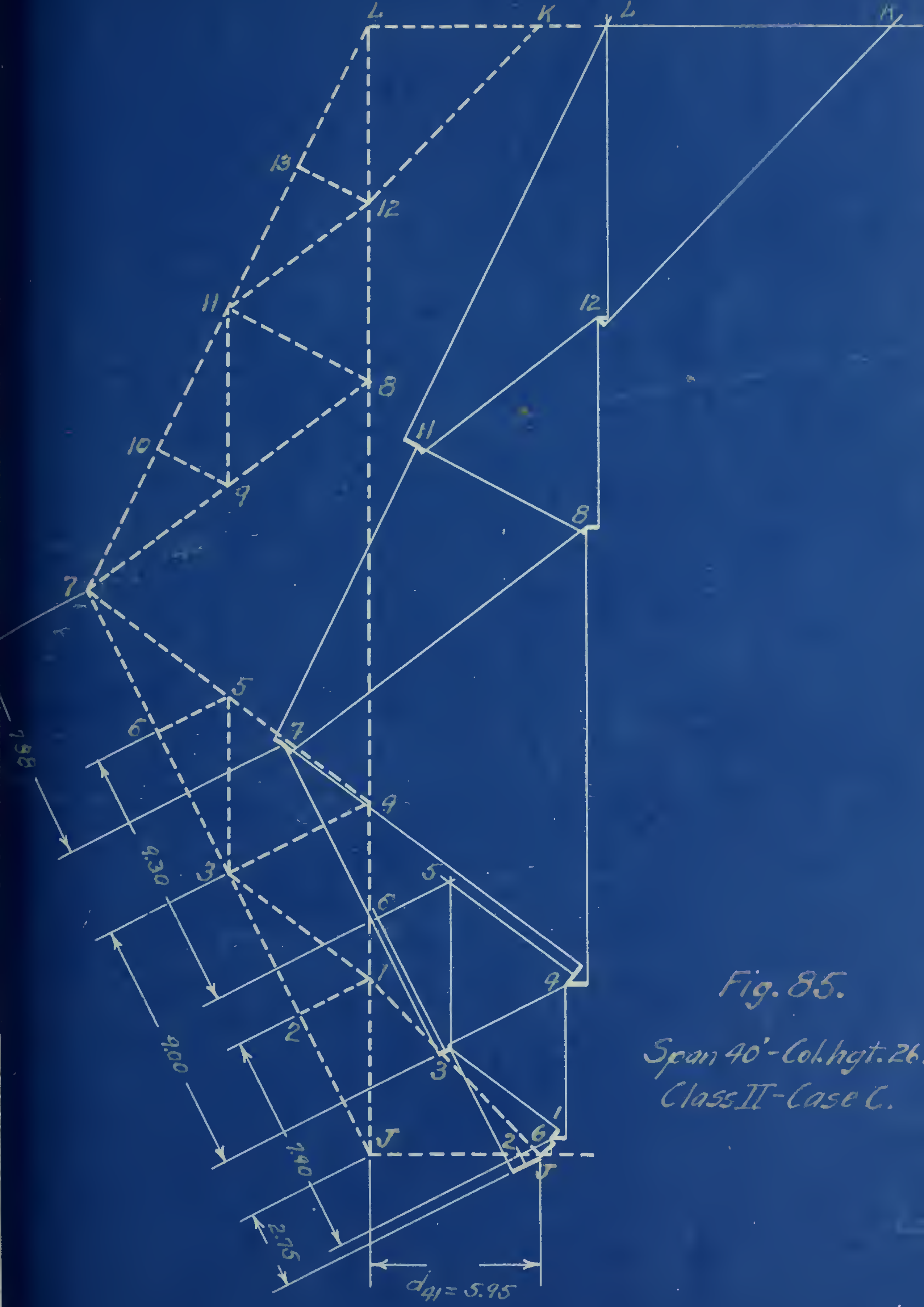


Fig. 85.

Span 40'-Col. hgt. 26'.
Class II-Case C.

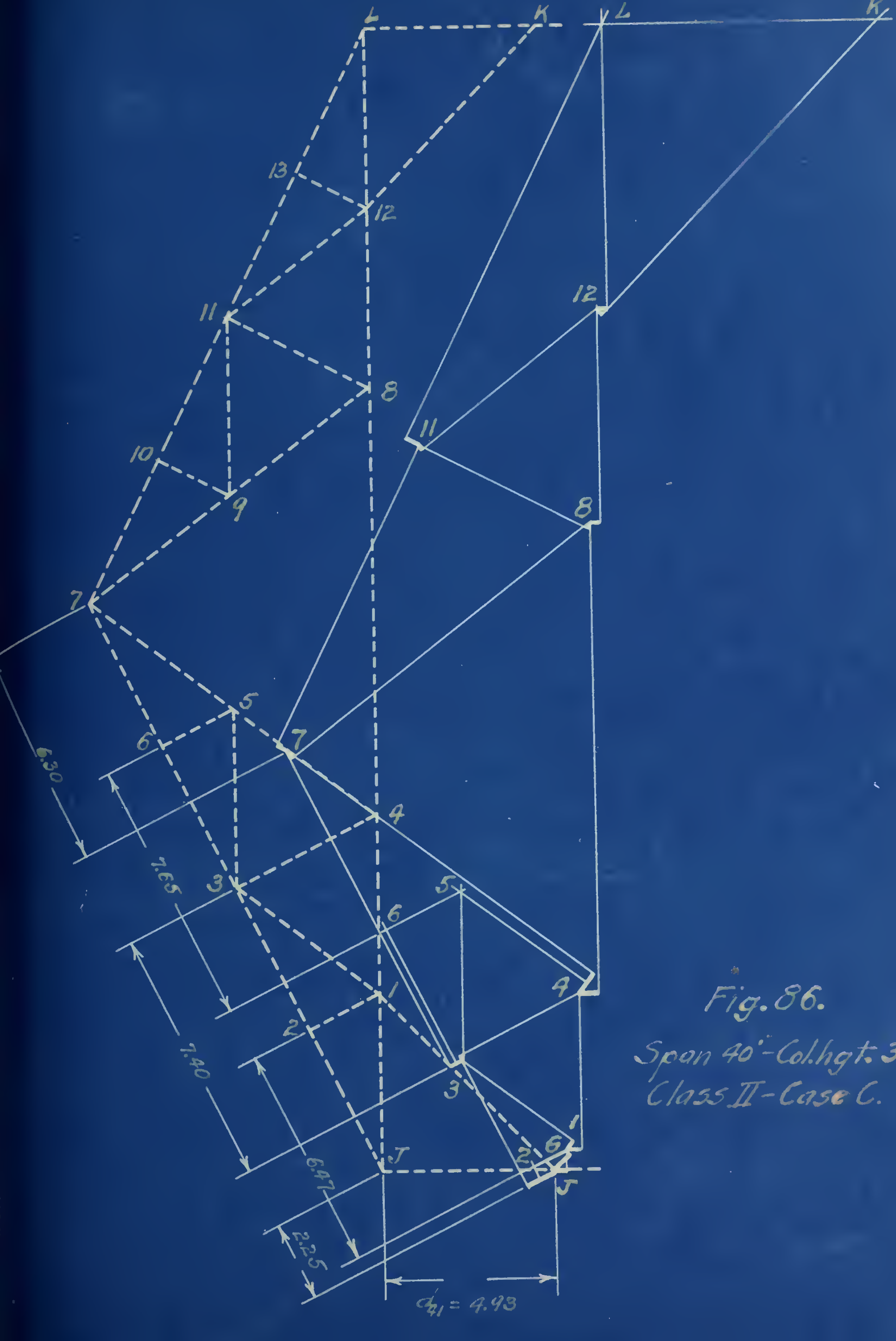


Fig. 86.

Span 40'-Col. hgt. 31'.

Class II-Case C.

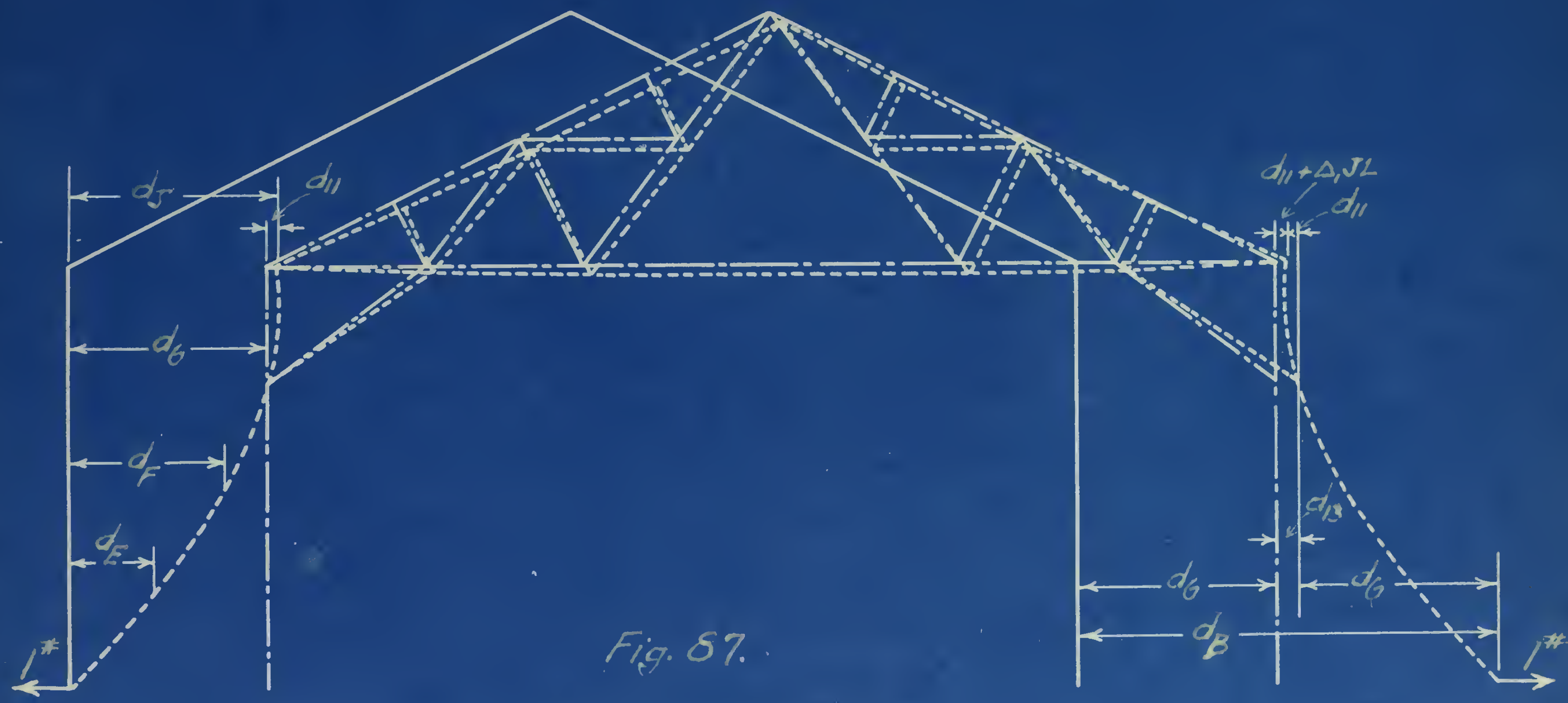


Fig. 87.

Span 50'-Col. hgt. 21 ft.
Class I-Case A.

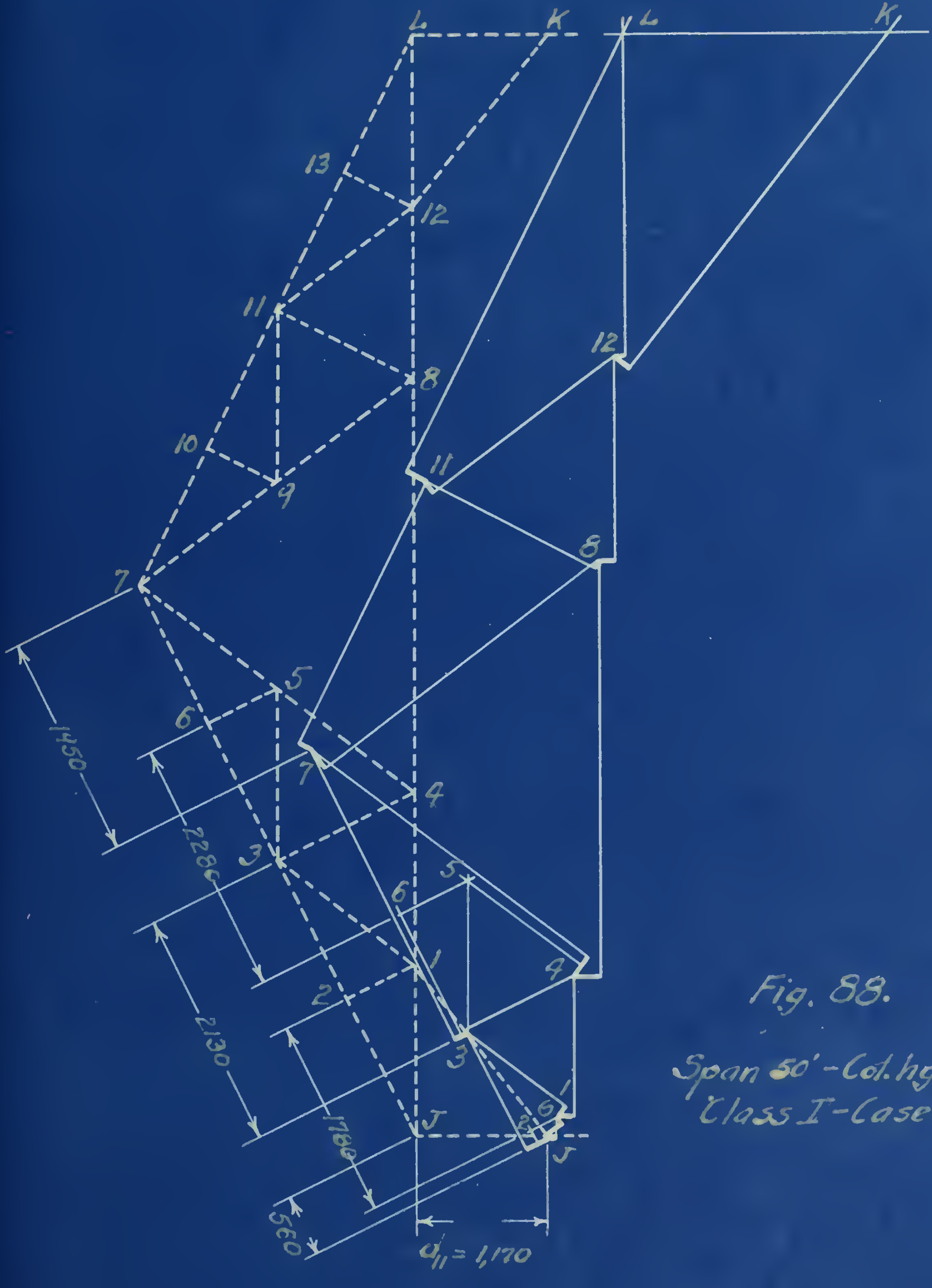


Fig. 88.

Span 50' - Col. hgt. 16'.
Class I - Case A.

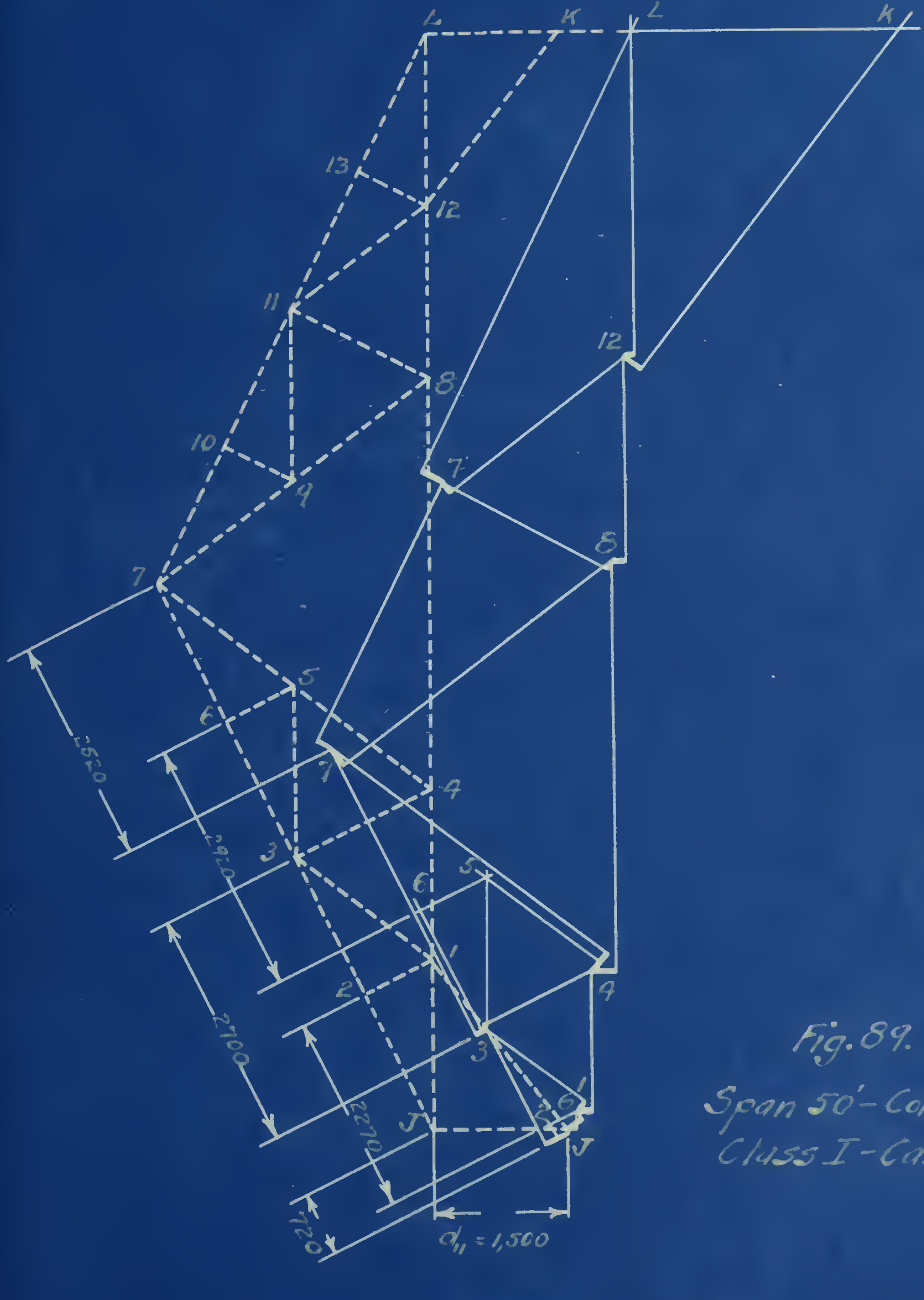


Fig. 89.

Span 50' - Col. Lgt. 21'.
Class I - Case A.

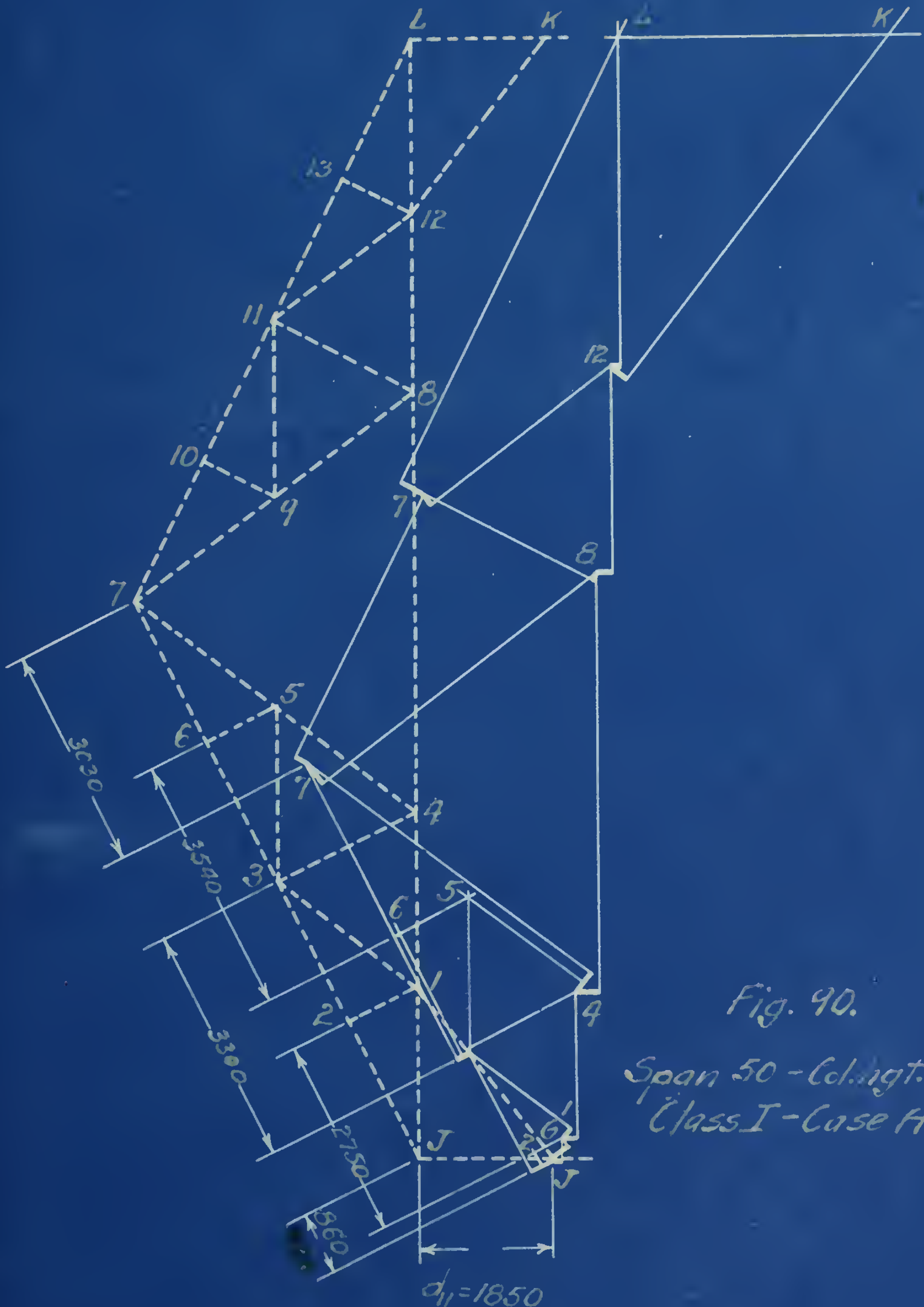


Fig. 90.

Span 50 - Col. hgt. 26'.
Class I - Case A.

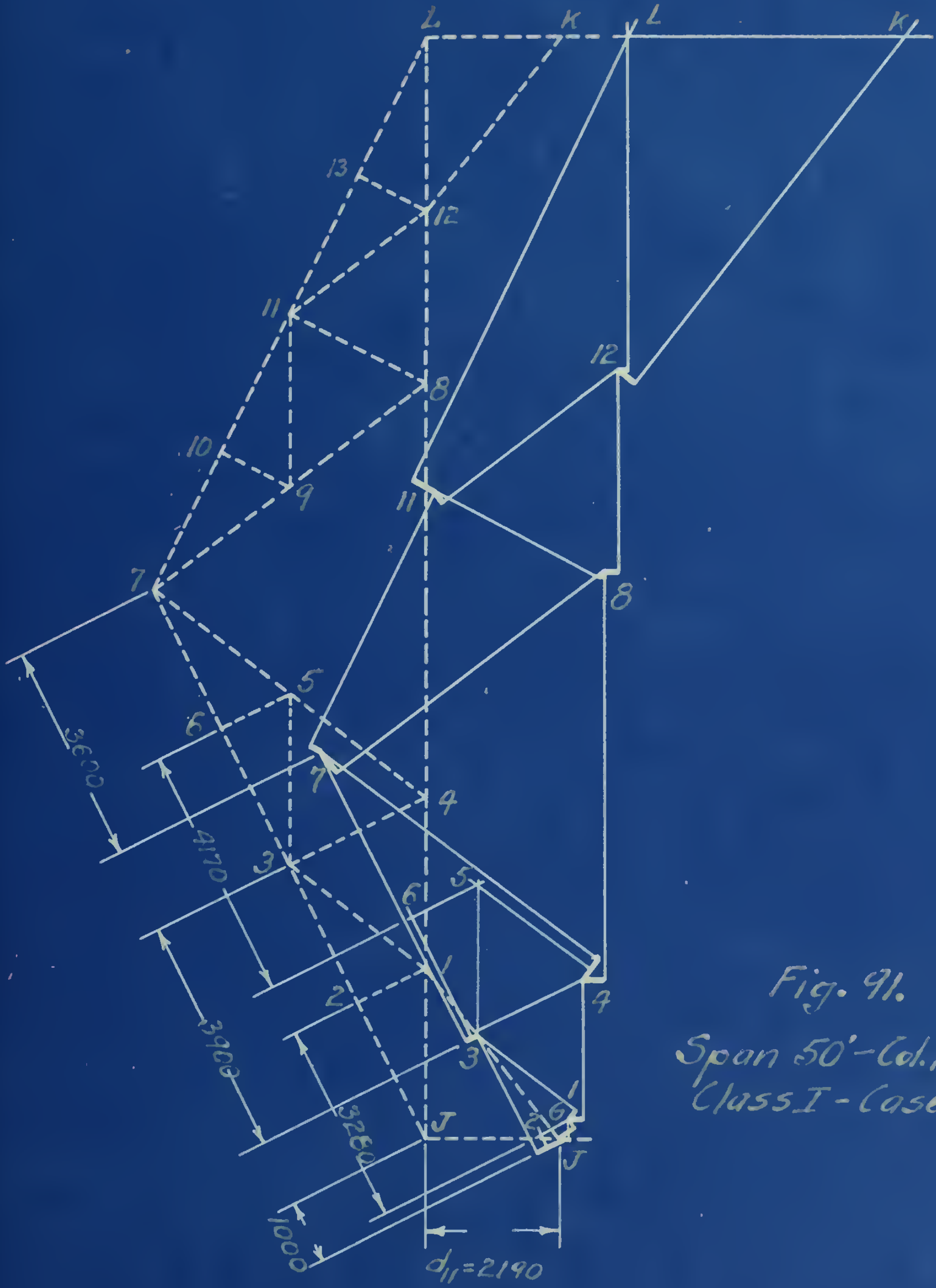


Fig. 91.
Span 50'-Col. hgt. 31'.
Class I - Case A.

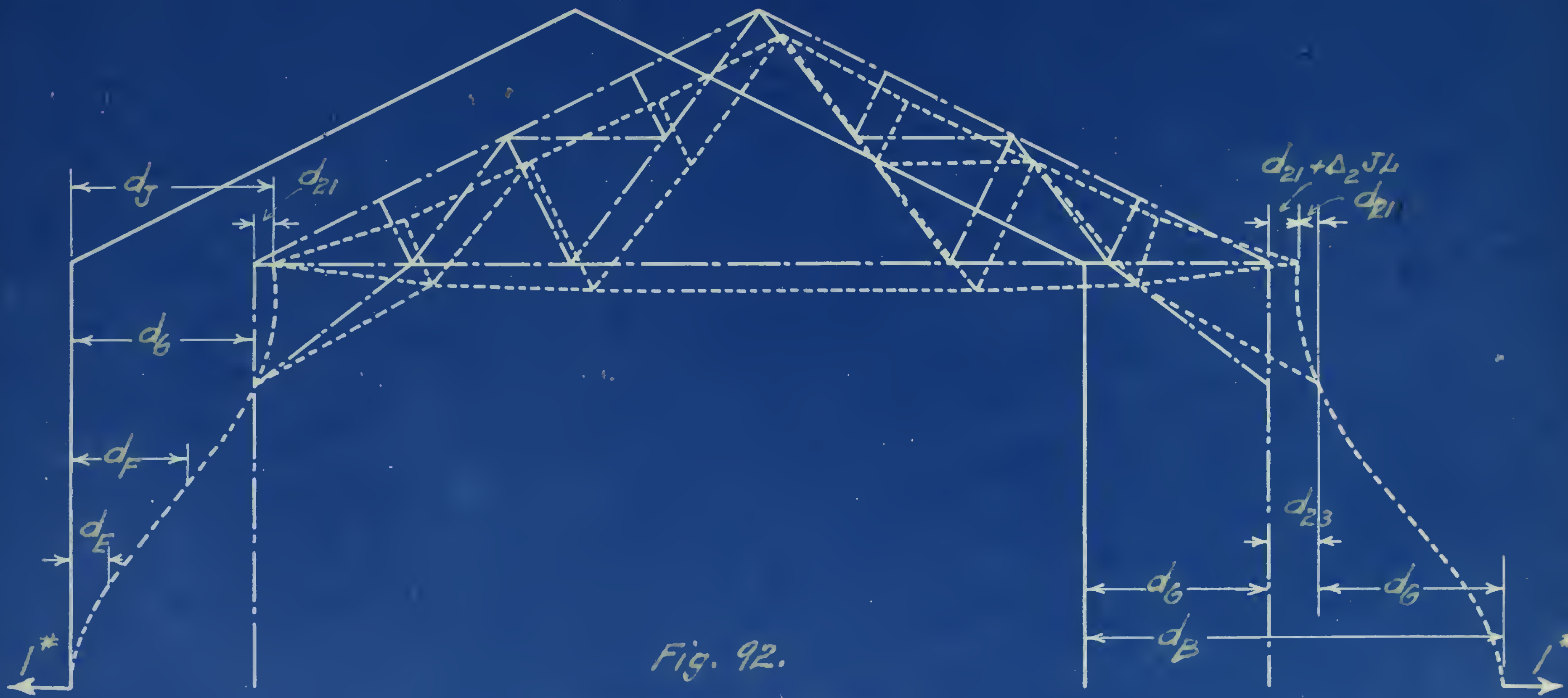


Fig. 92.

Span 50'-Col. hgt 21.'
Class II-Case A.

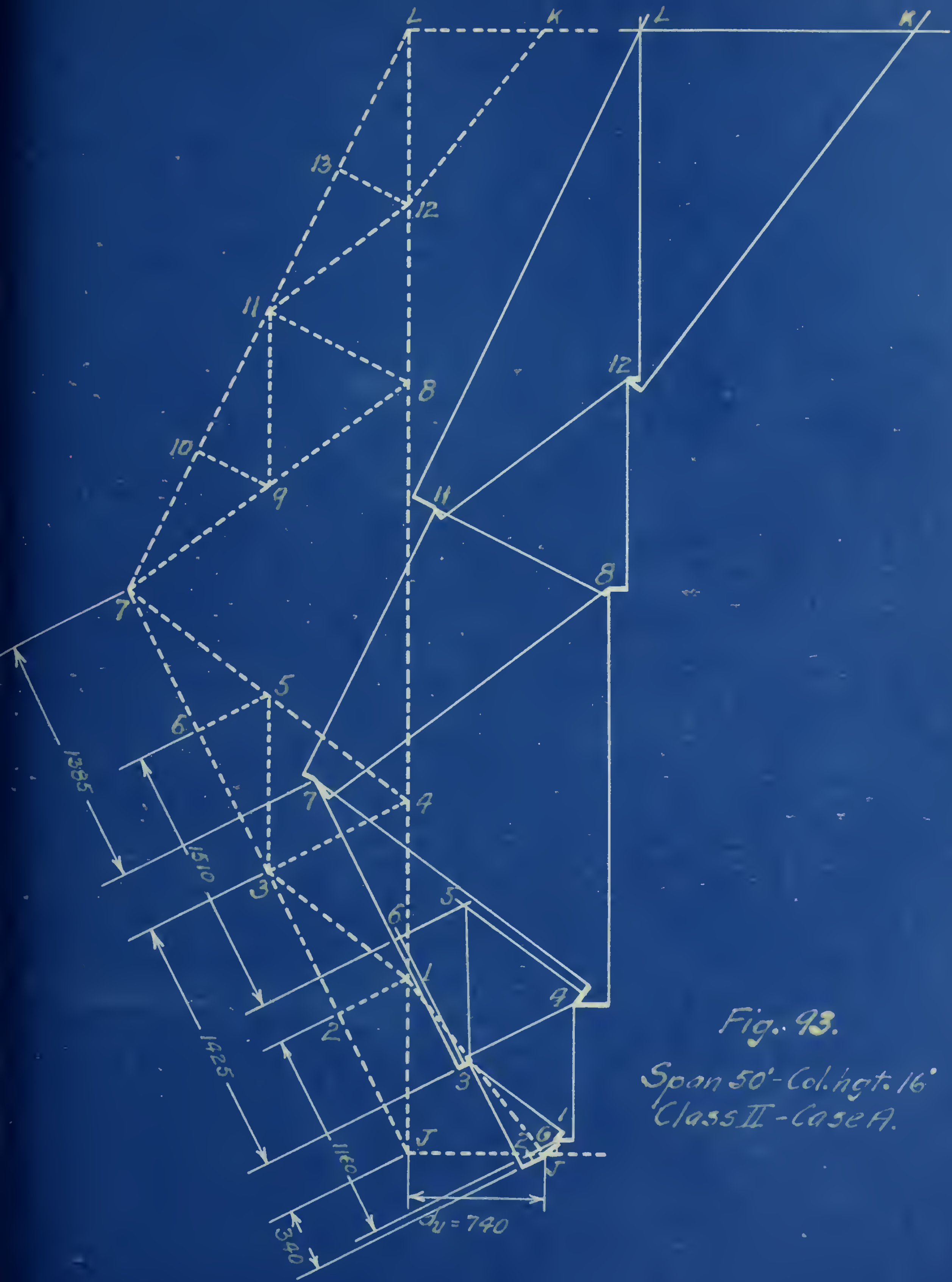


Fig. 93.

Span 50' - Col. hgt. 16'
Class II - Case A.

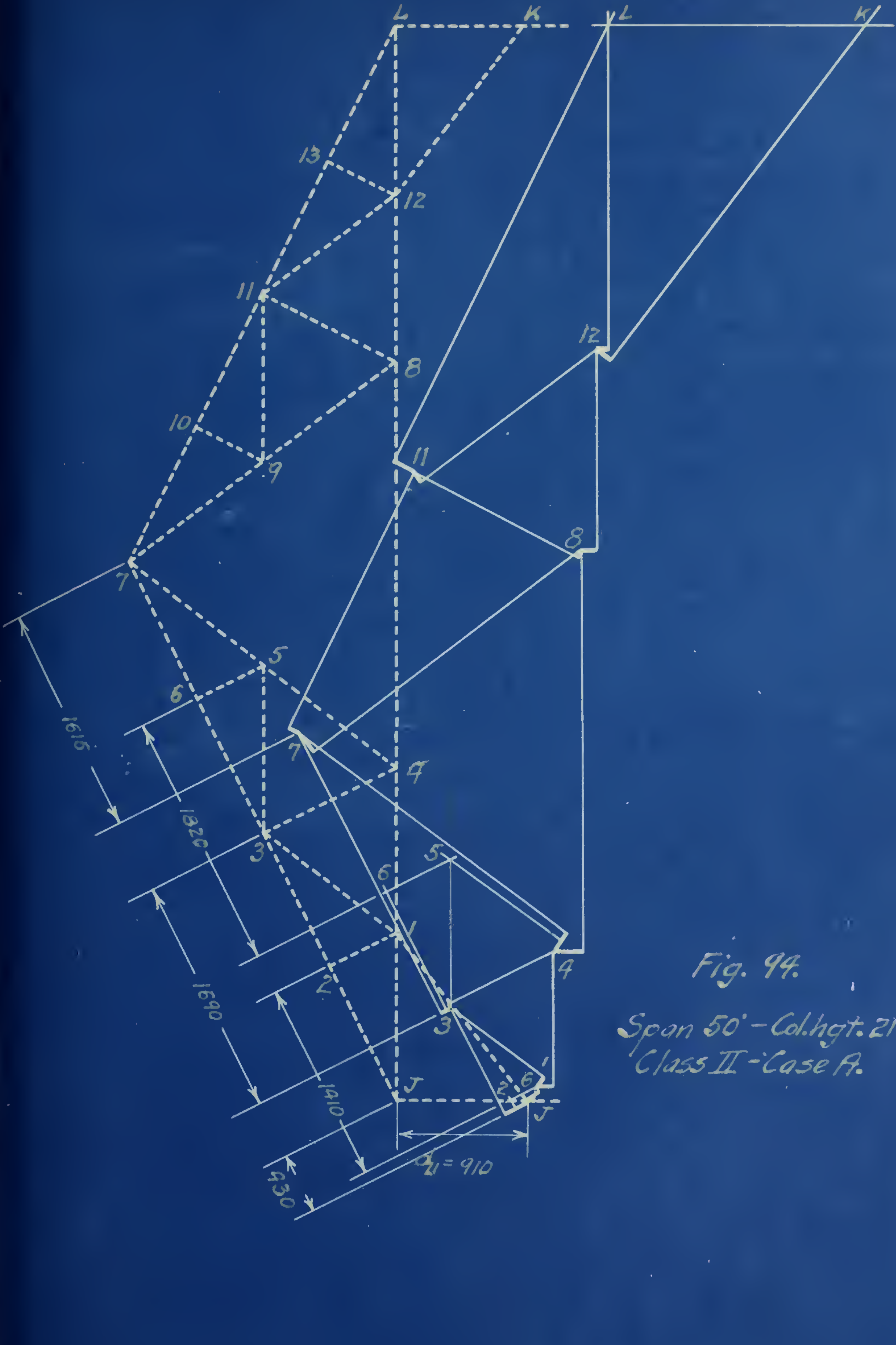


Fig. 94.

Span 50' – Col. hgt. 21'.
Class II – Case A.

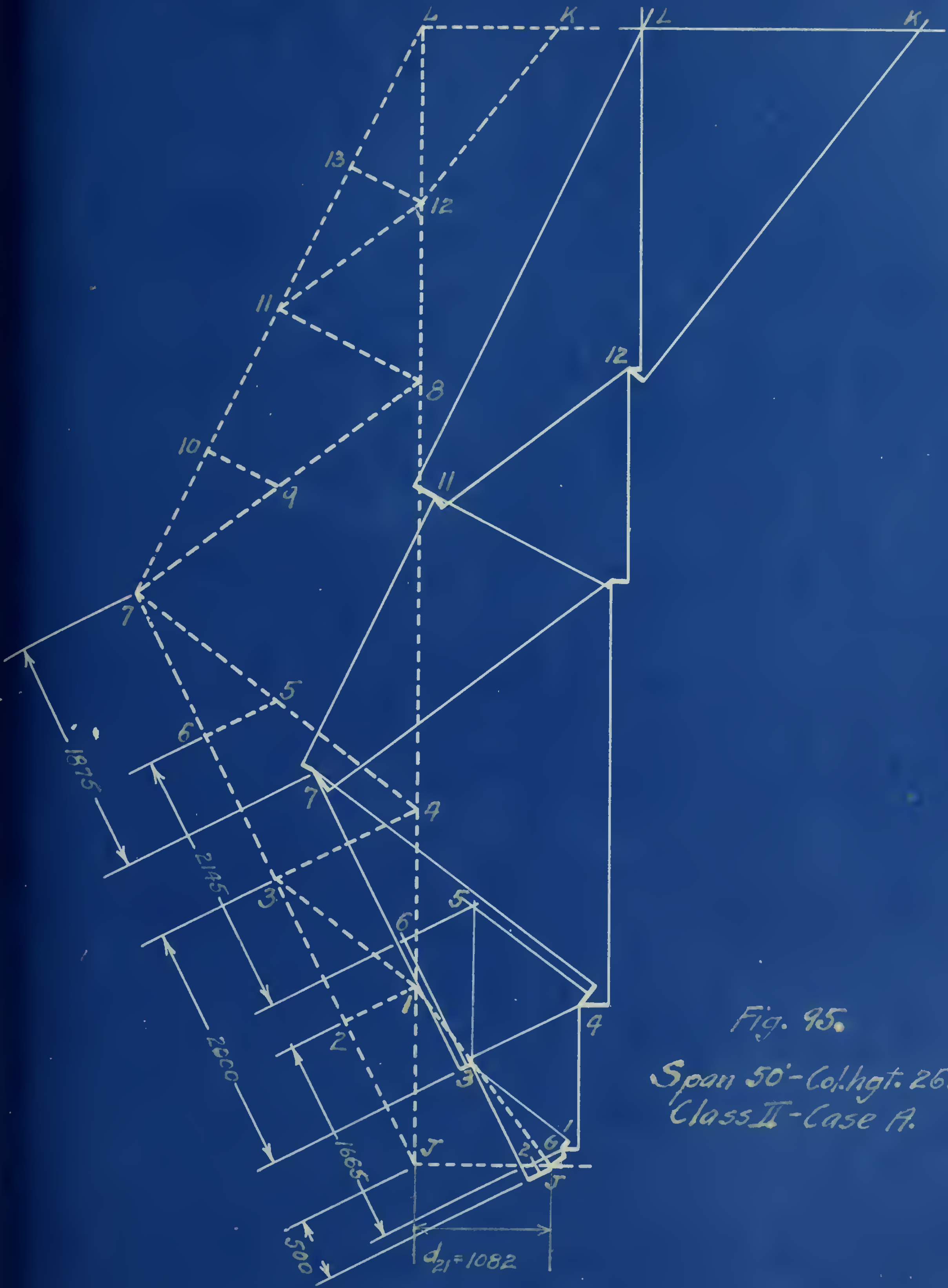


Fig. 95.

Span 50'-Col. hgt. 26'.
Class II - Case A.

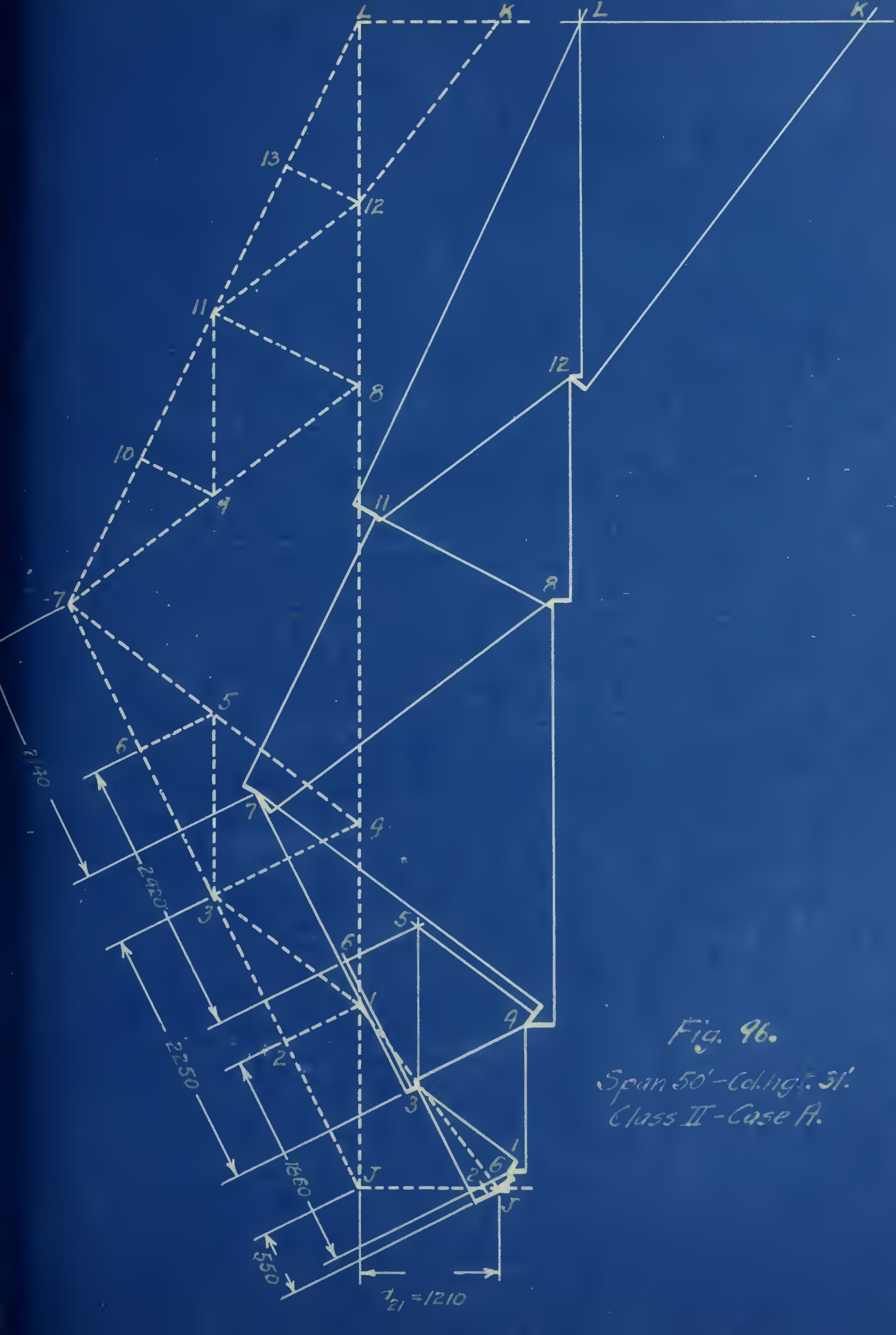


Fig. 96.

Span 50'-Col.hg.: 31'.
Class II-Case A.

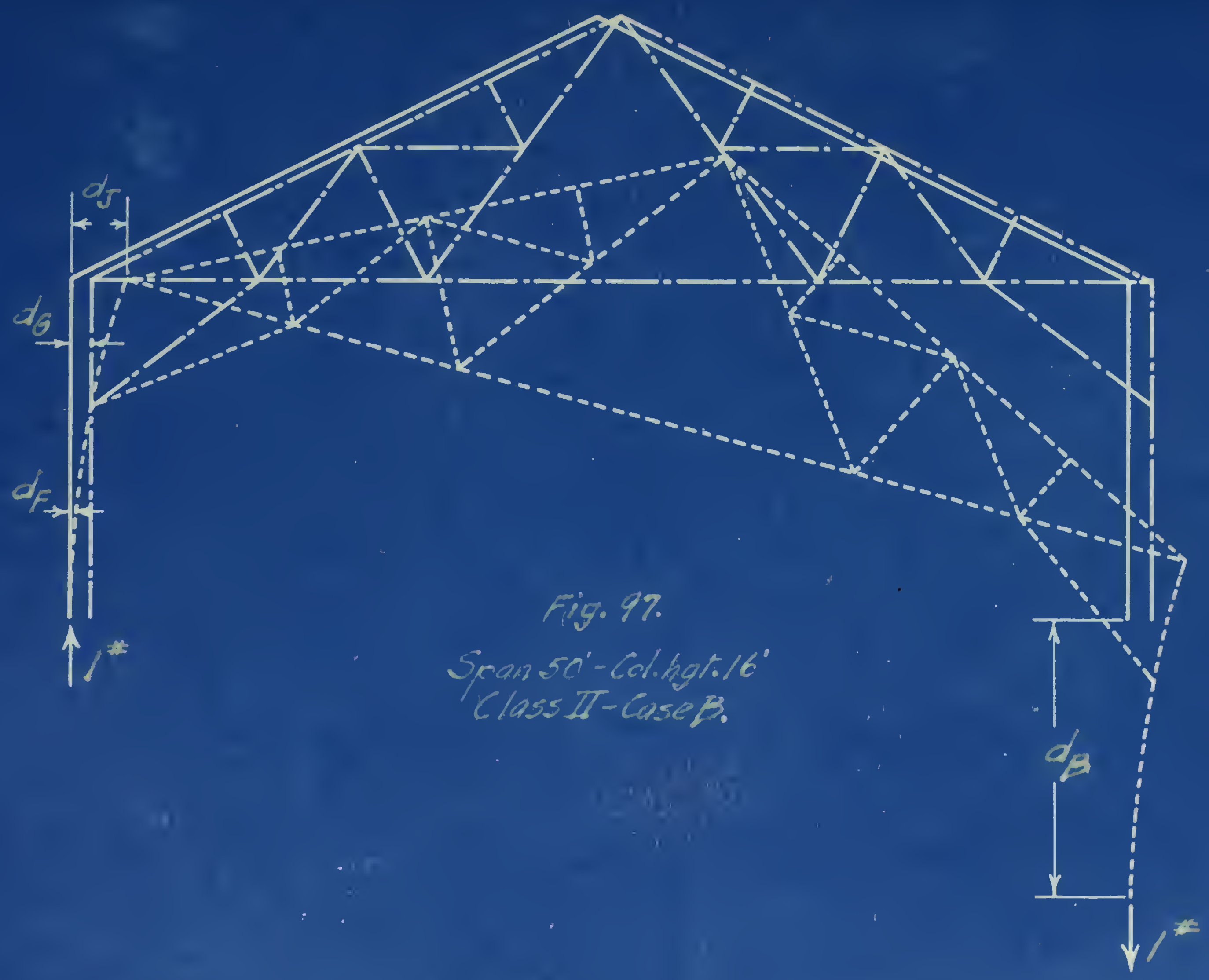


Fig. 97.

Span 50'-Col. hgt. 16'
Class II-Case B.

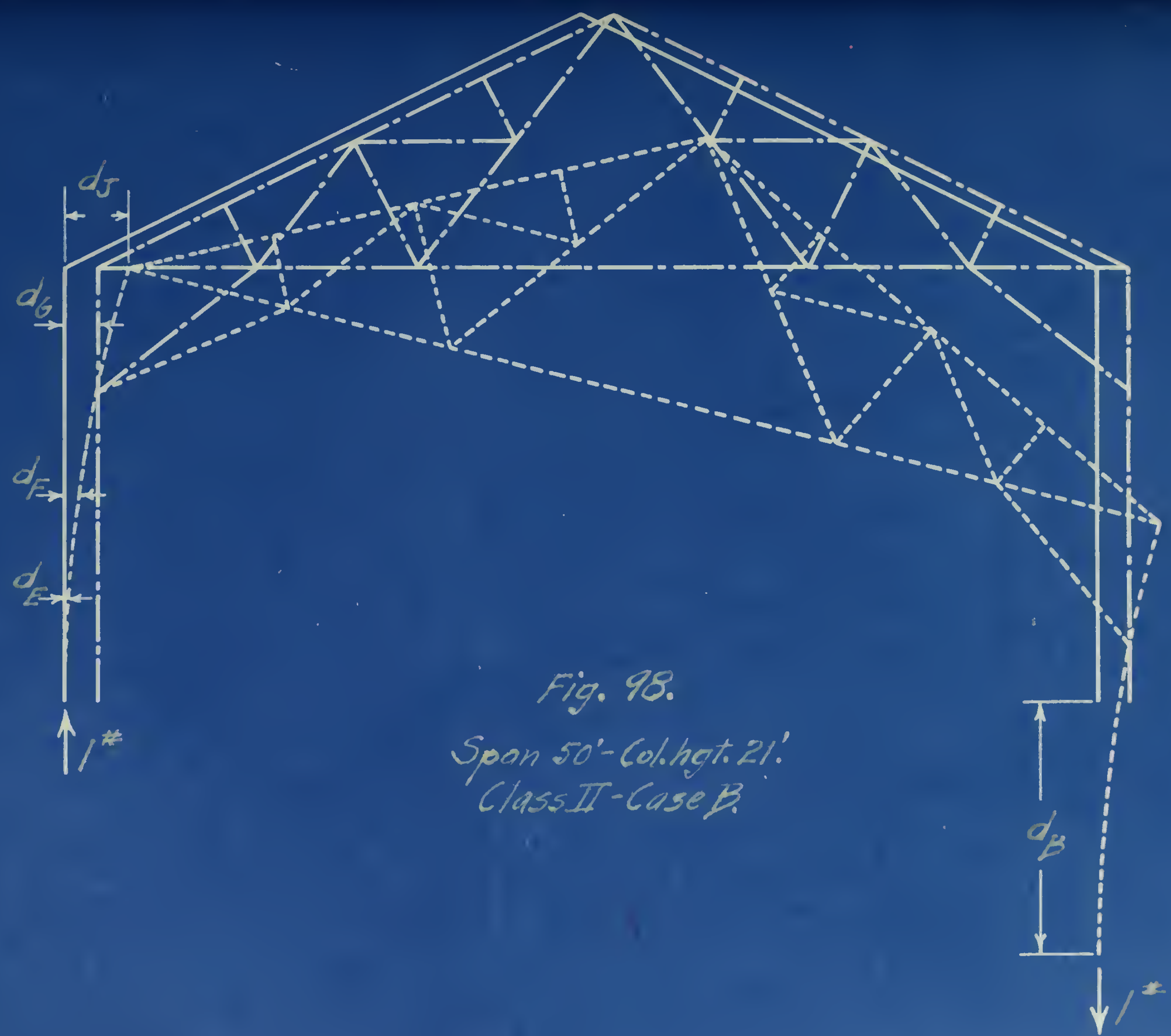


Fig. 98.

Span 50'-Col. hgt. 21'.
Class II-Case B.

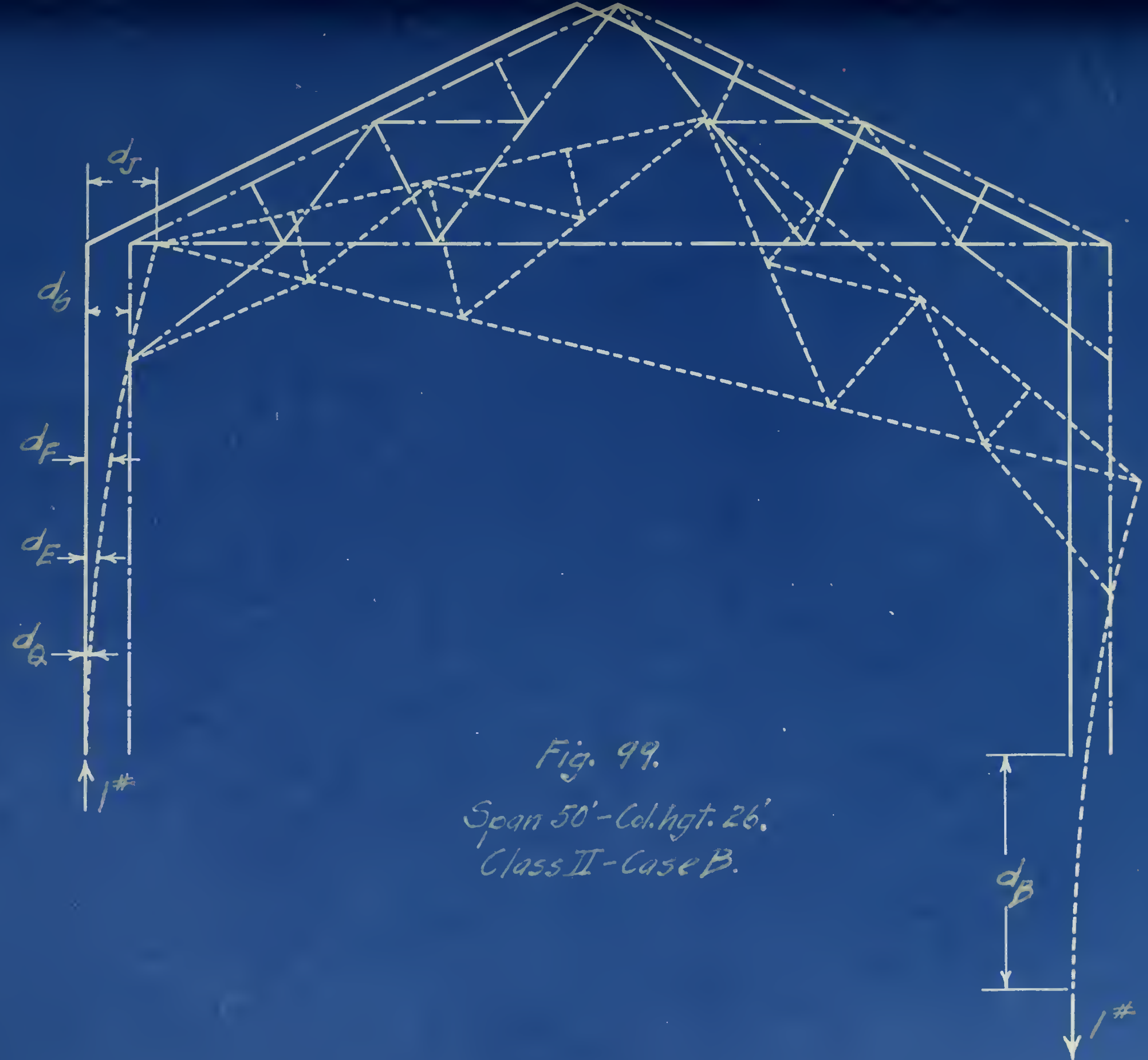


Fig. 99.

Span 50'-Col. hgt. 26'.
Class II-Case B.

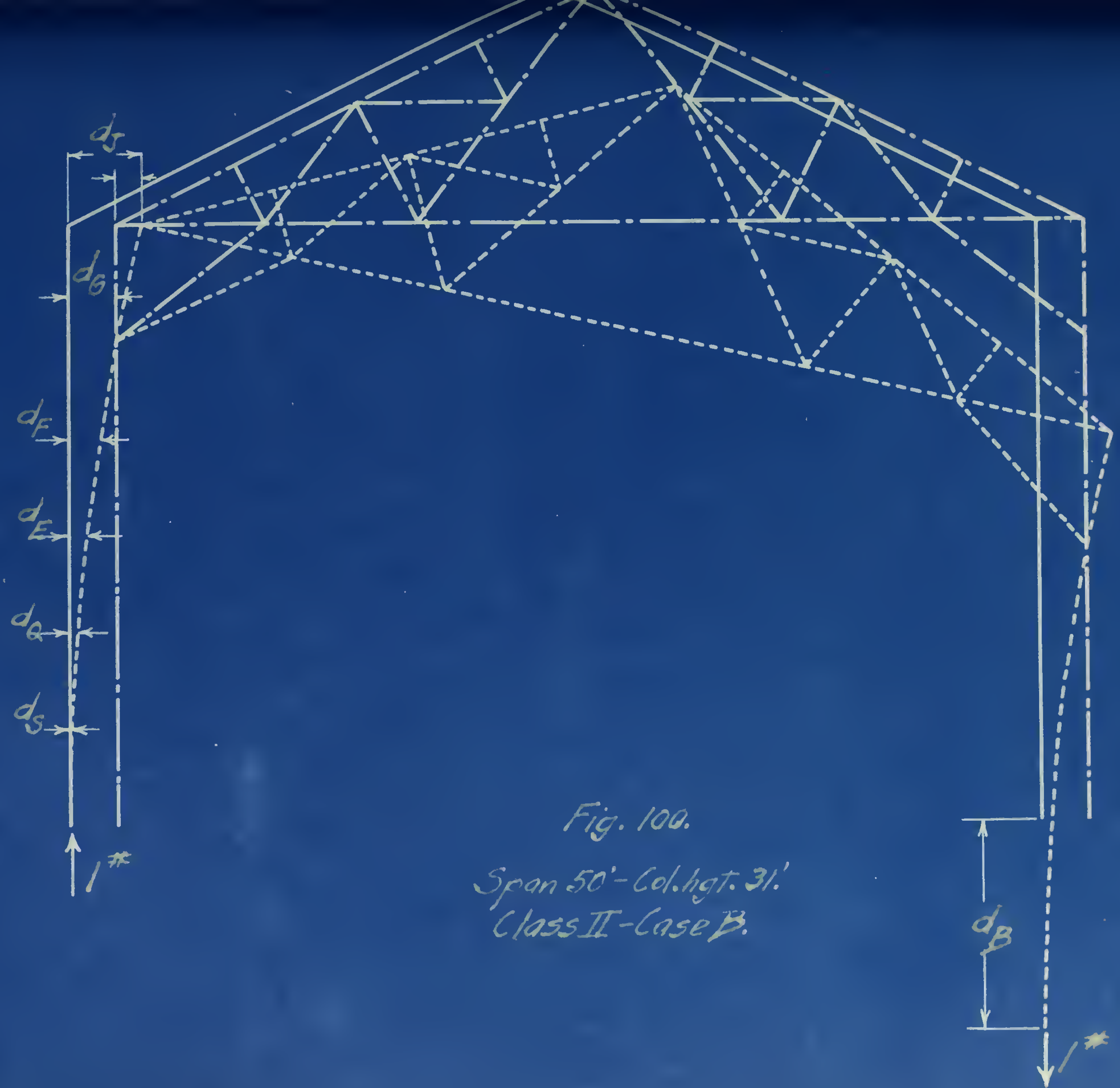


Fig. 100.

Span 50'-Col. hgt. 31'.
Class II-Case B.

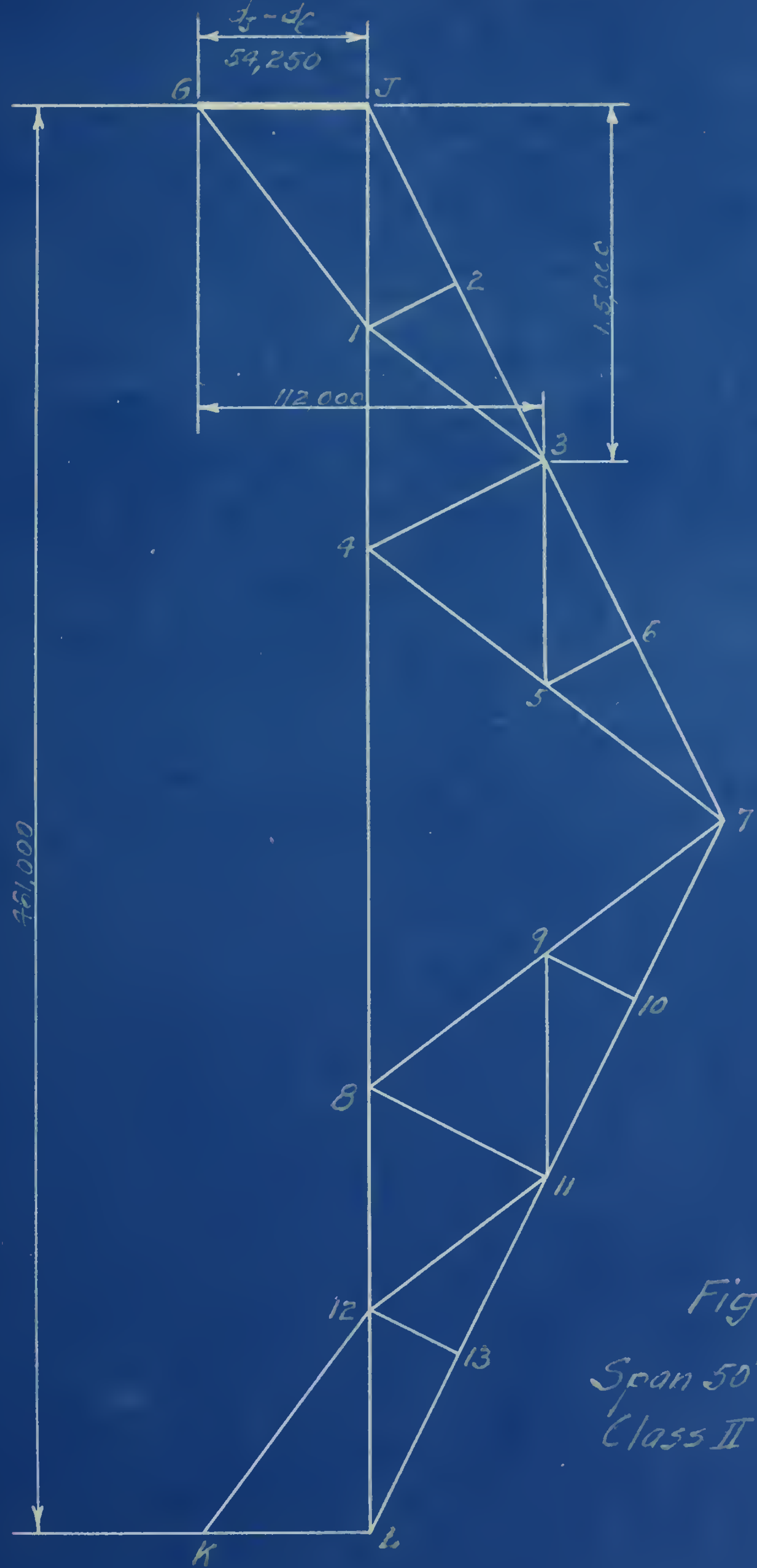


Fig. 101.

Span 50' - Col. hgt. 21'.
Class II - Case B.

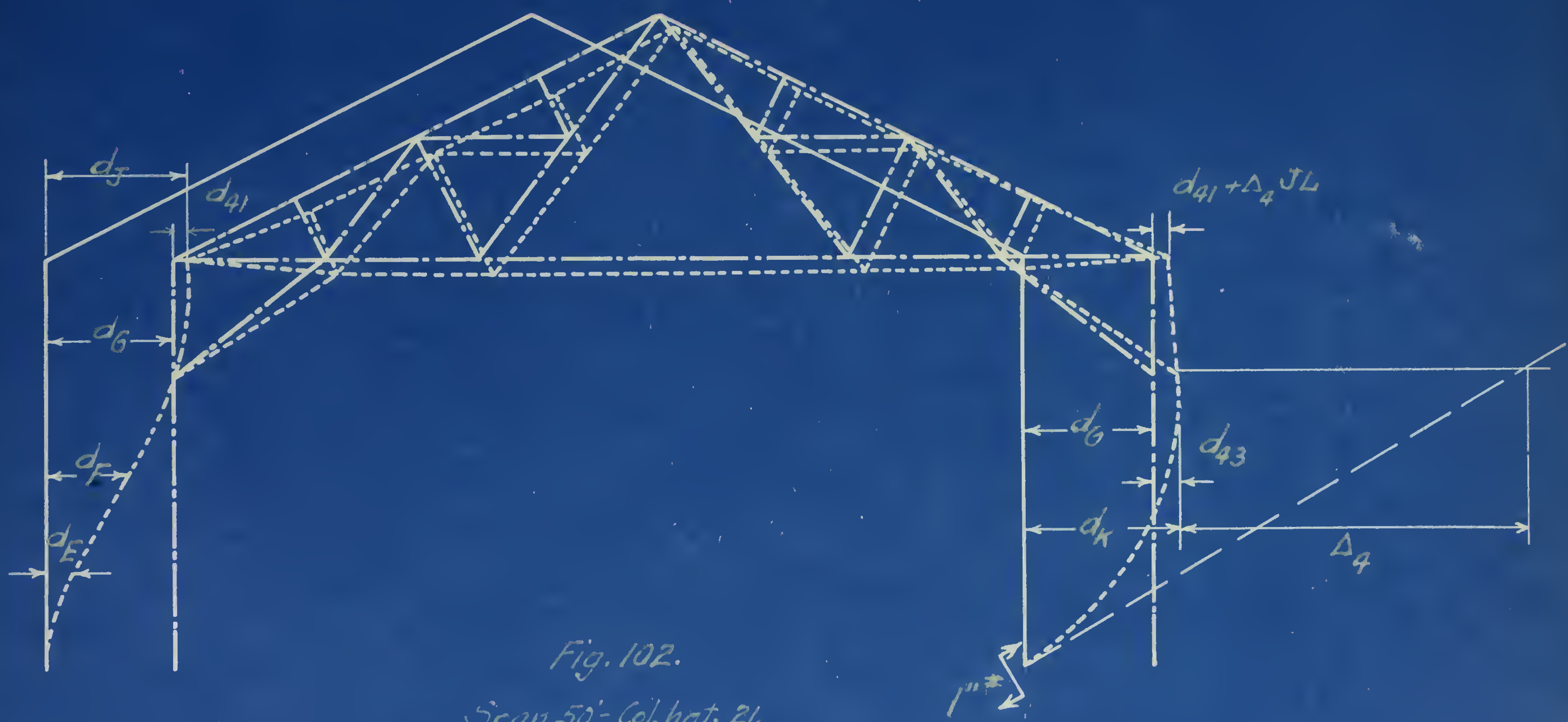


Fig. 102.

Span 50' - Col. hgt. 21.
Class II - Case C.

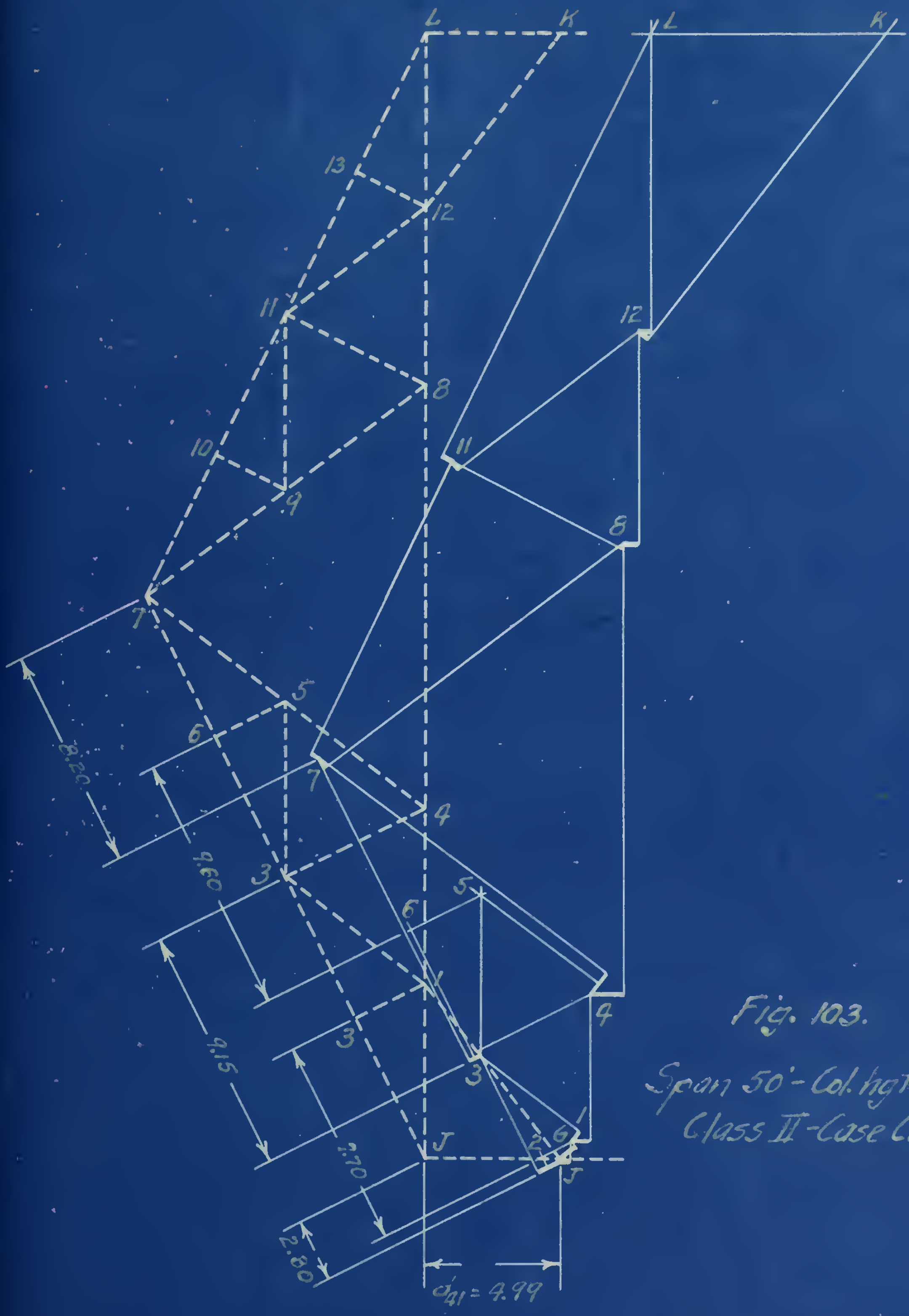


Fig. 103.

Span 50' - Col. hgt. 16

Class II - Case C.

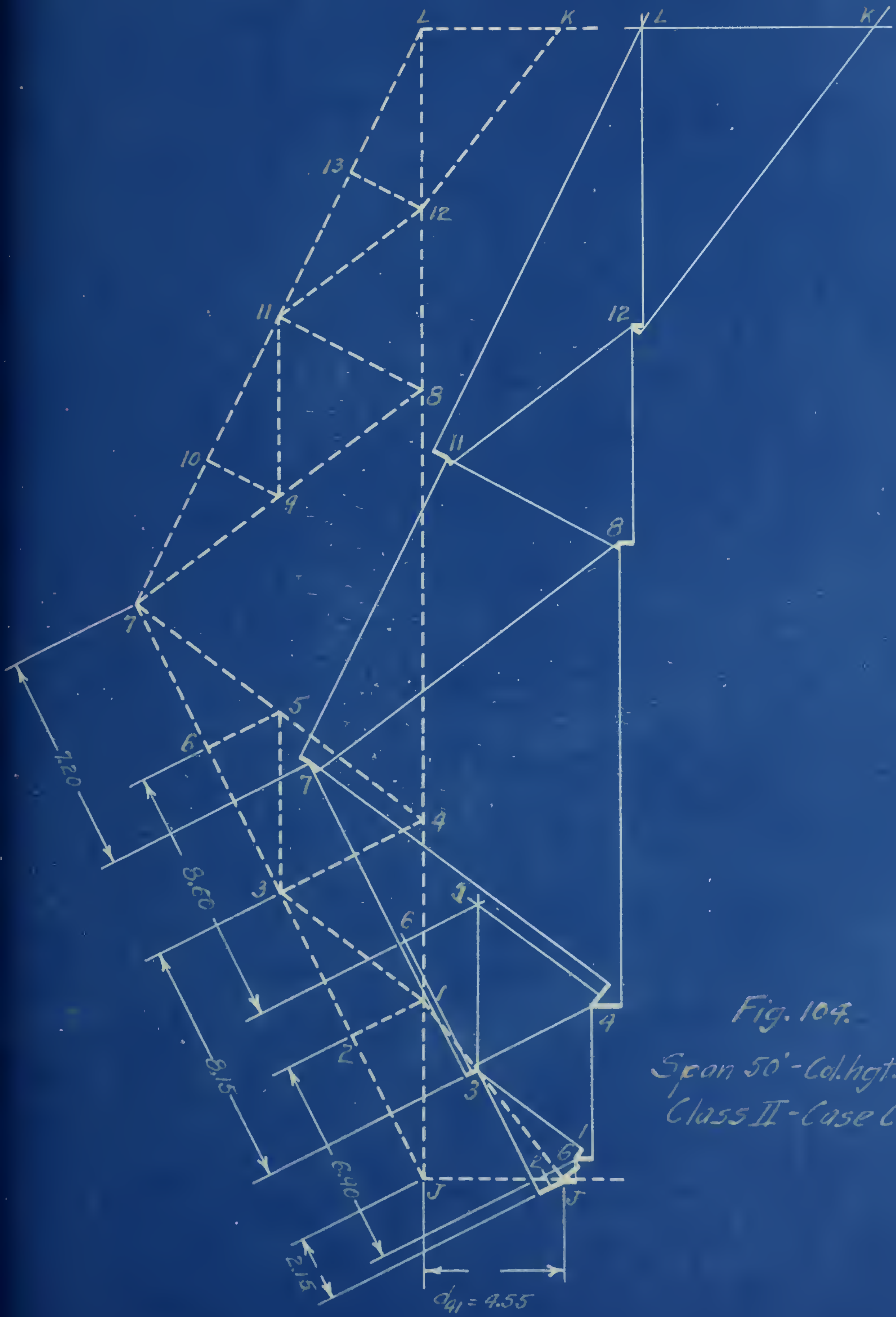


Fig. 104.

Span 50'-Col. hgt. 21'.
Class II-Case C.

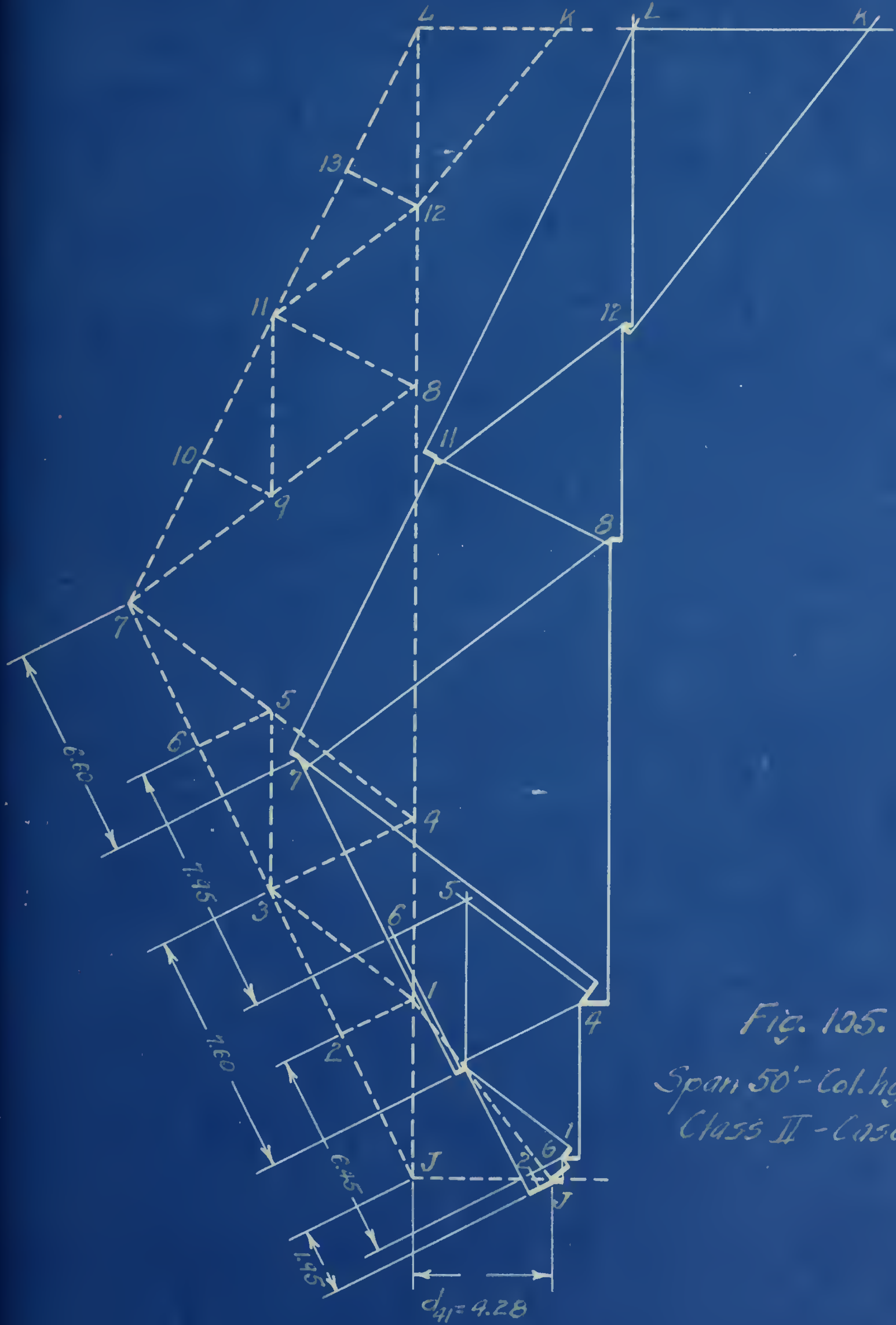


Fig. 125.
Span 50' - Col. hgt. 26'.
Class II - Case C.

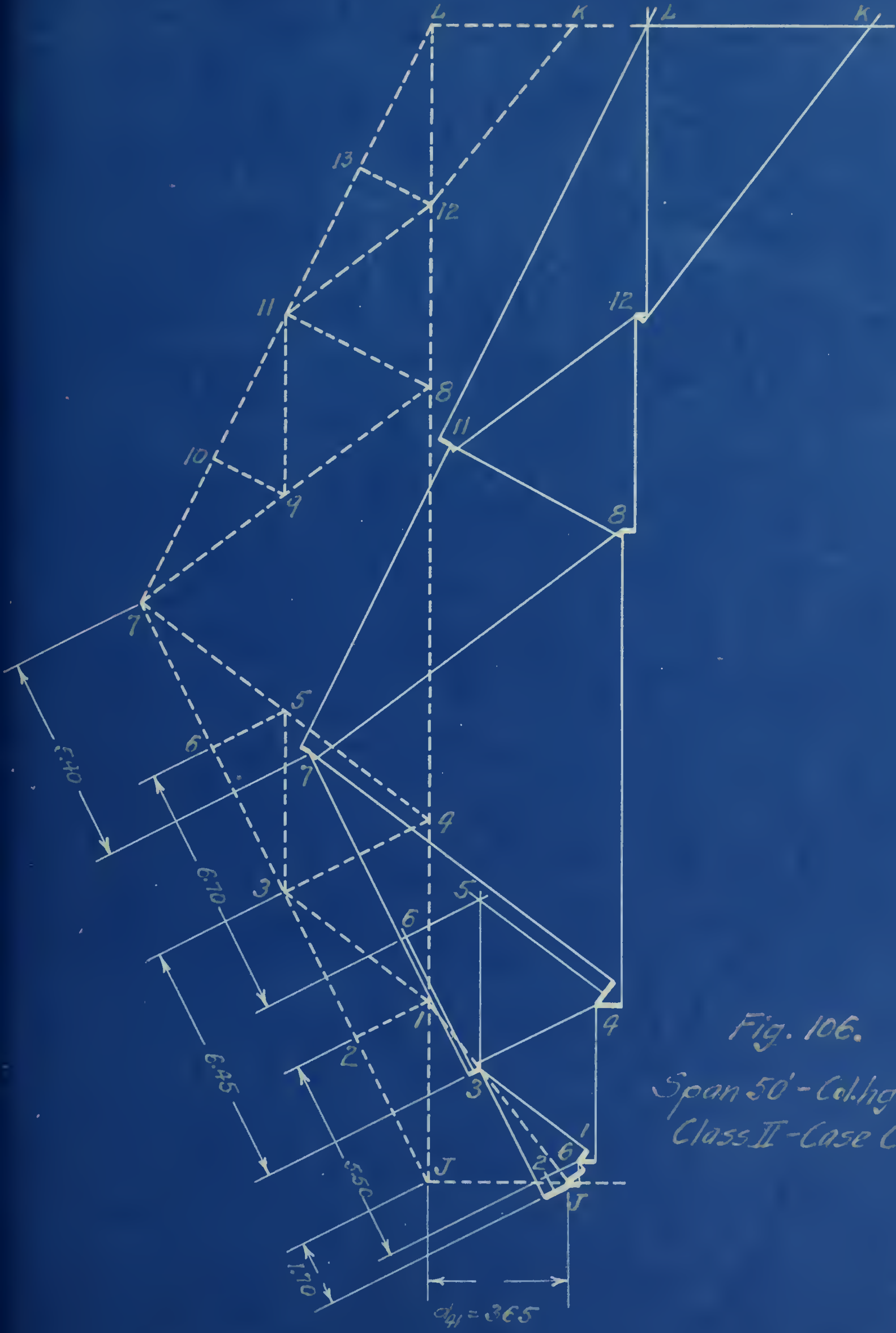


Fig. 106.

Span 50' - Col. hgt. 31'.

Class II - Case C.

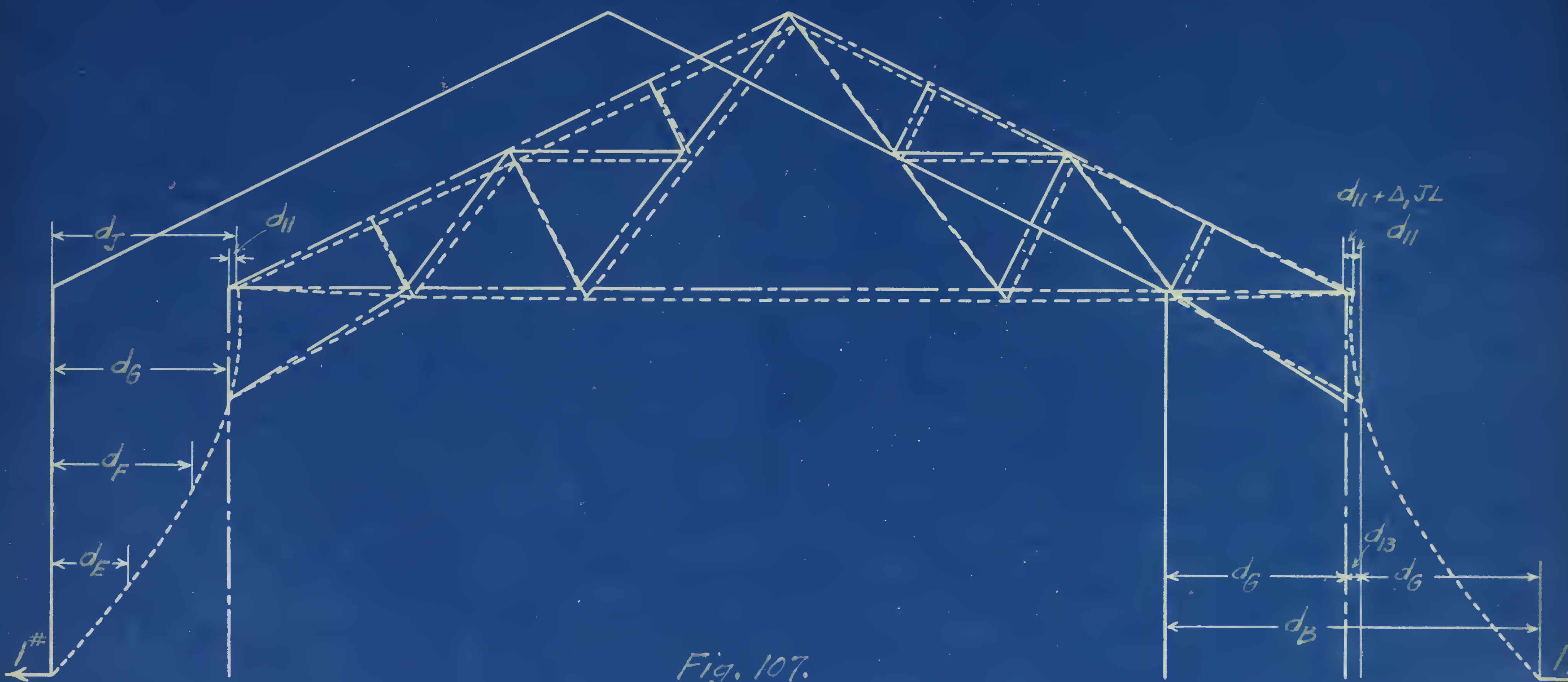


Fig. 107.

Span 60' – Col. hgt. 21 ft.
Class I – Case A.

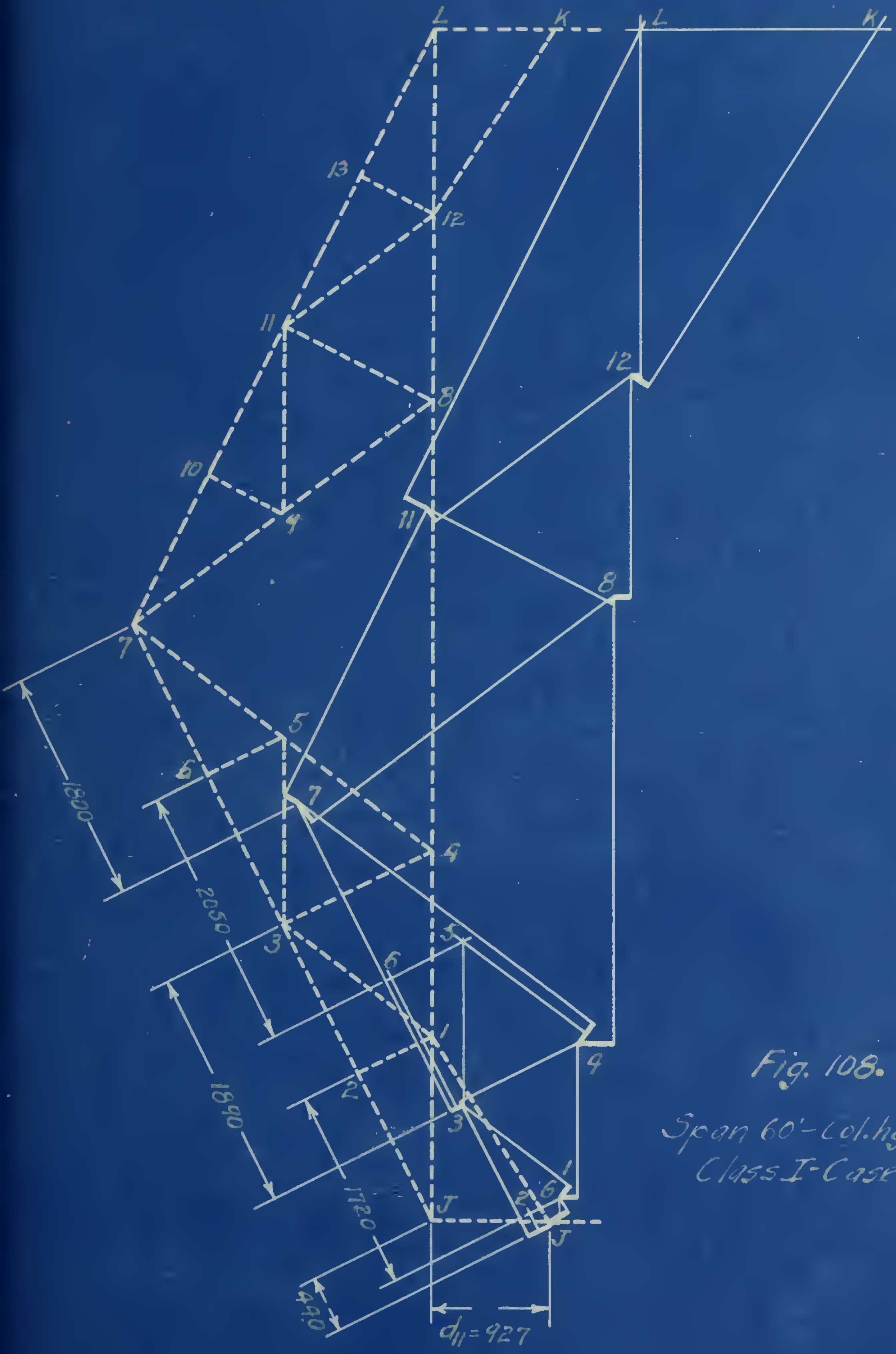


Fig. 108.

Span 60'-Col. hgt. 16'.
Class I-Case A.

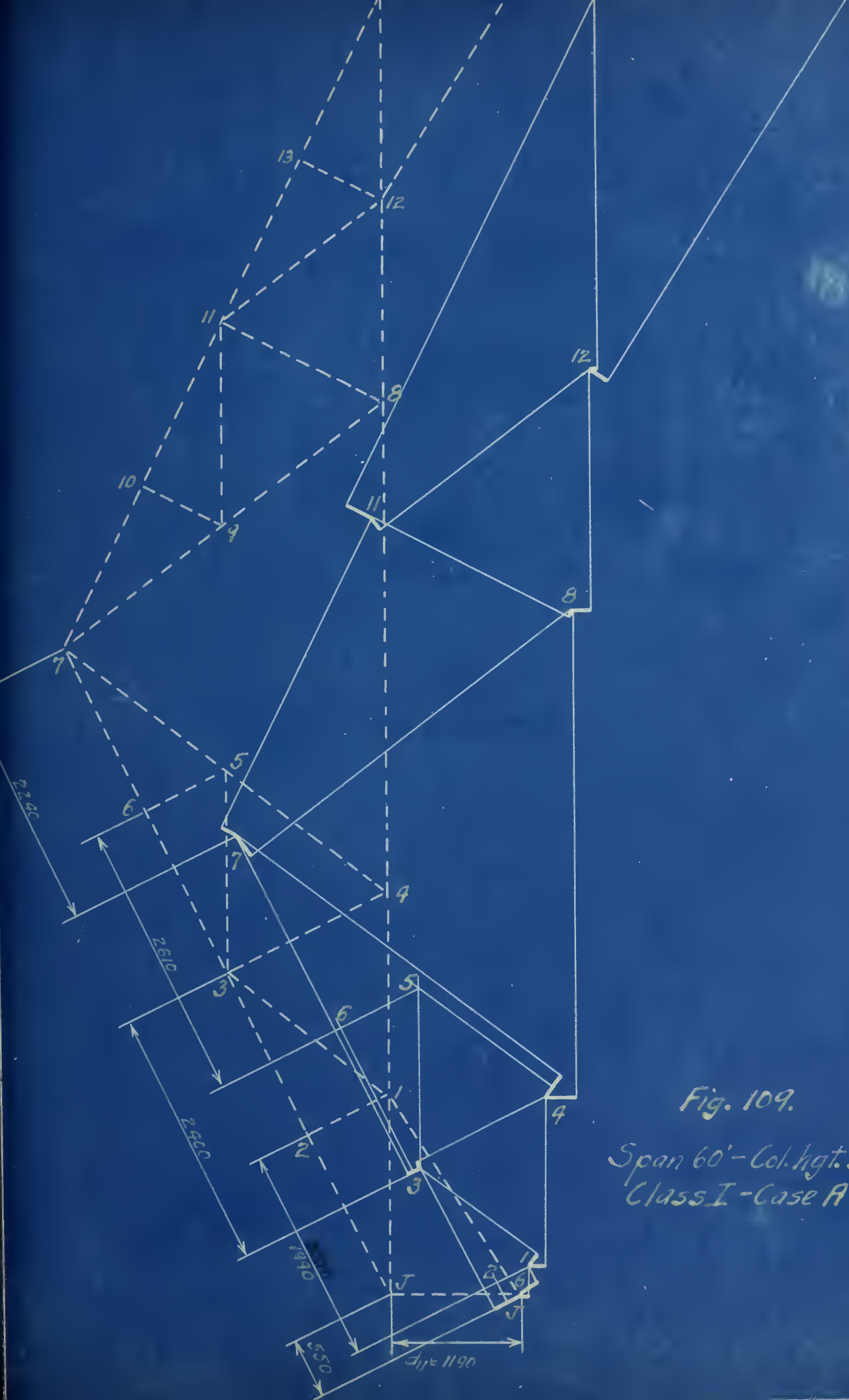


Fig. 109.
Span 60'-Col. hgt. 21'.
Class I-Case A

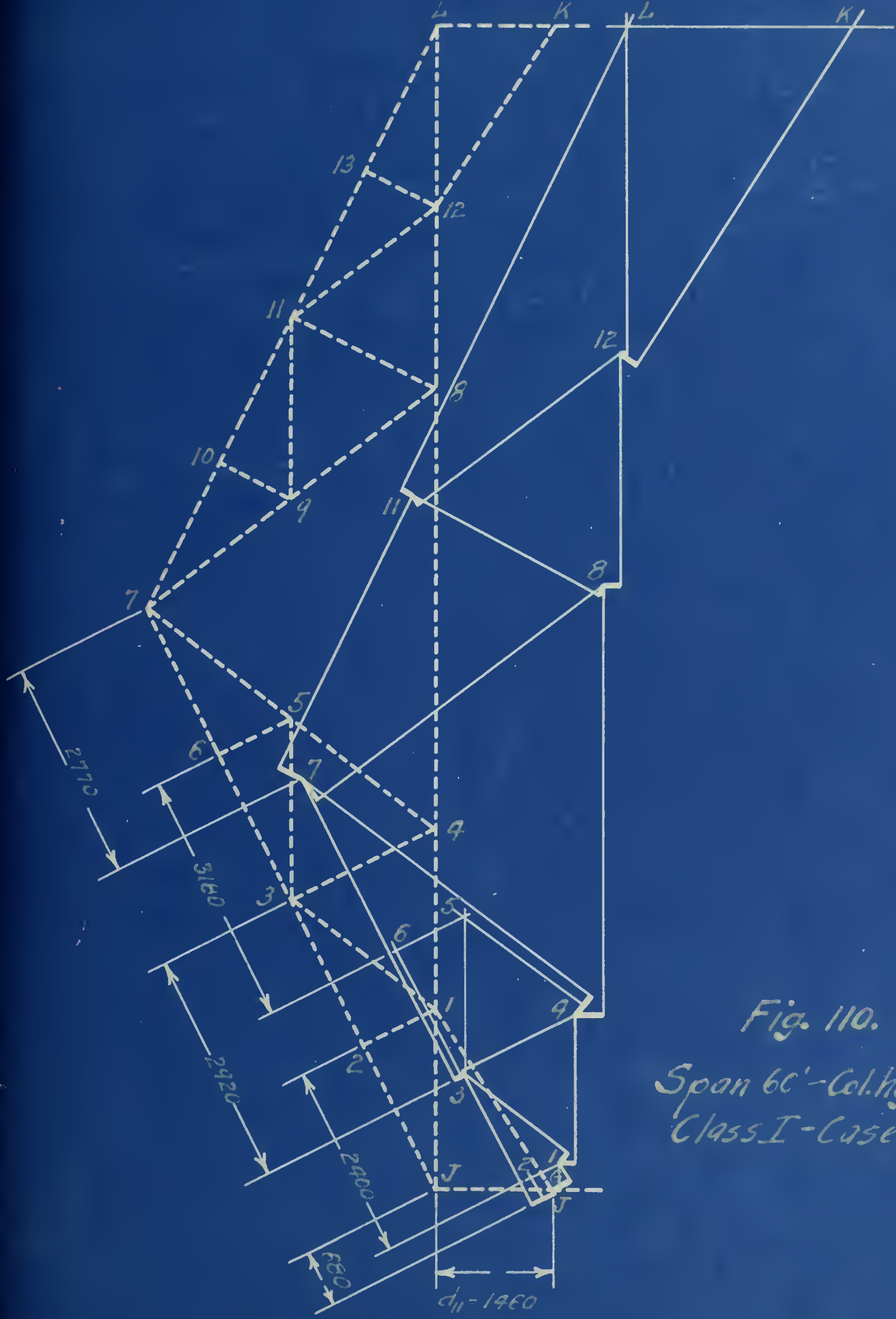


Fig. 110.

Span 60' - Col. hgt. 26'
Class I - Case A.

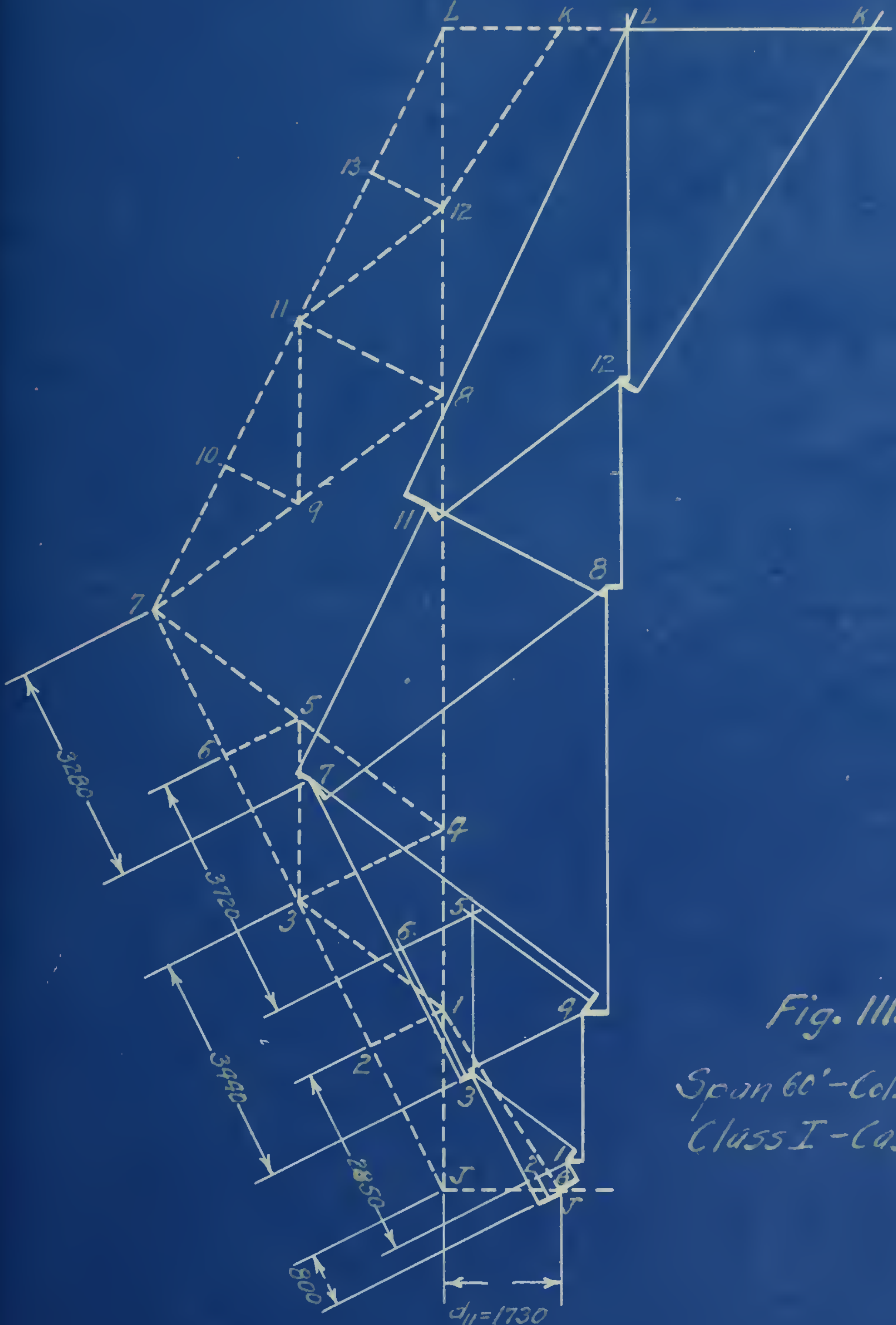


Fig. III.

Span 60' - Col. hgt. 31'.
Class I - Case A.

[illegible]
[illegible]
[illegible]

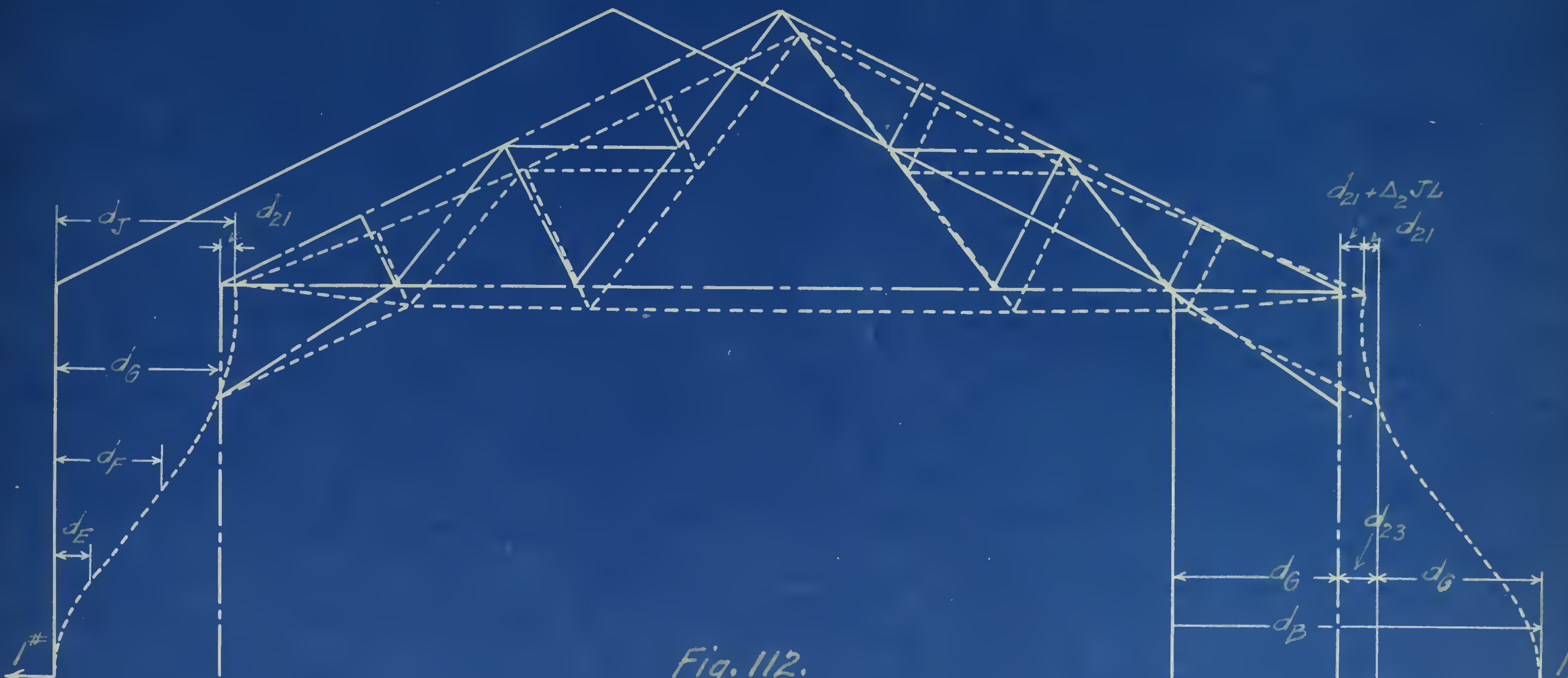


Fig. 112.

Span 60' - Col. hgt. 21'.
Class II - Case A.

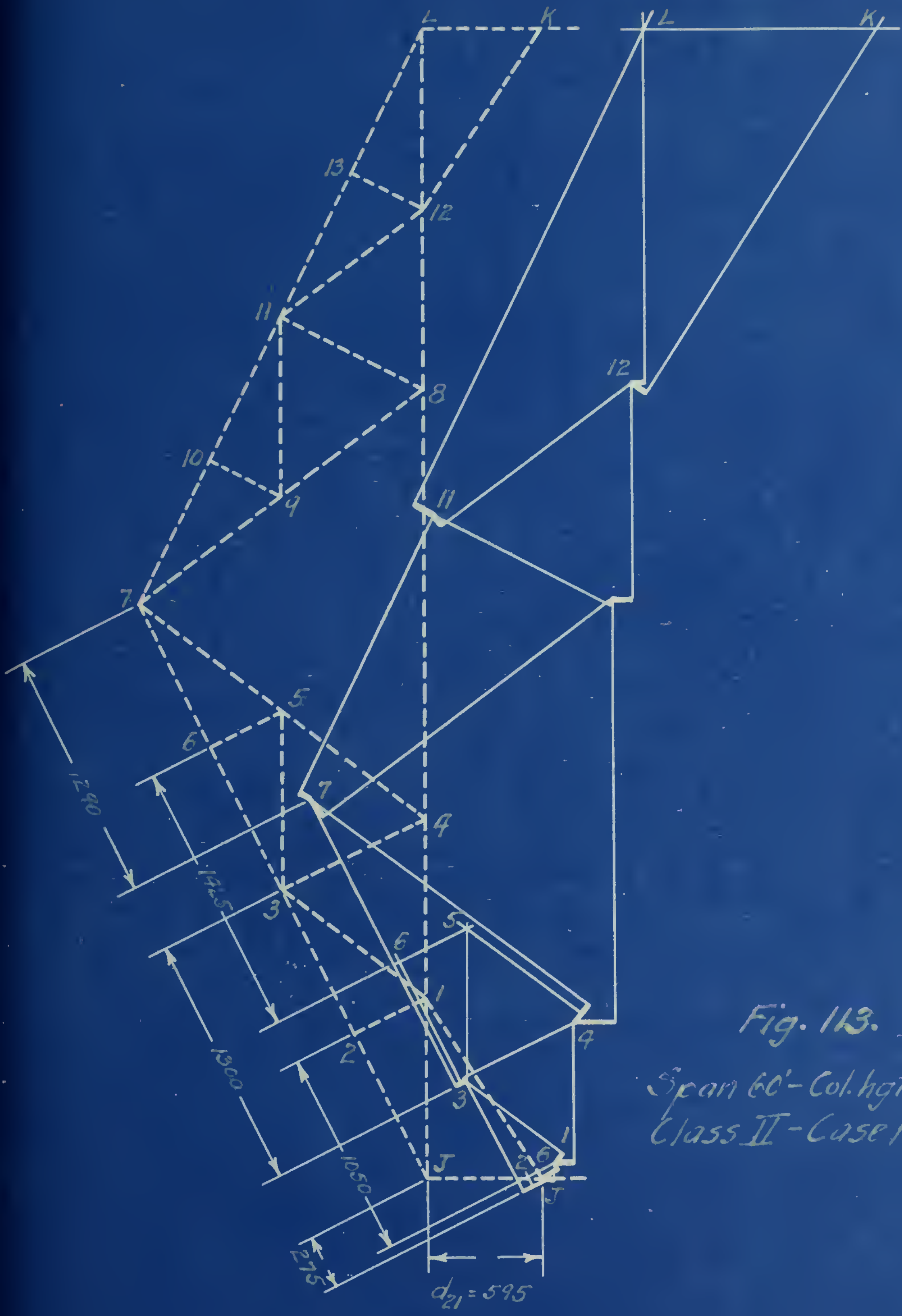


Fig. 113.

Span 60'-Col. hgt. 16'
Class II-Case A.

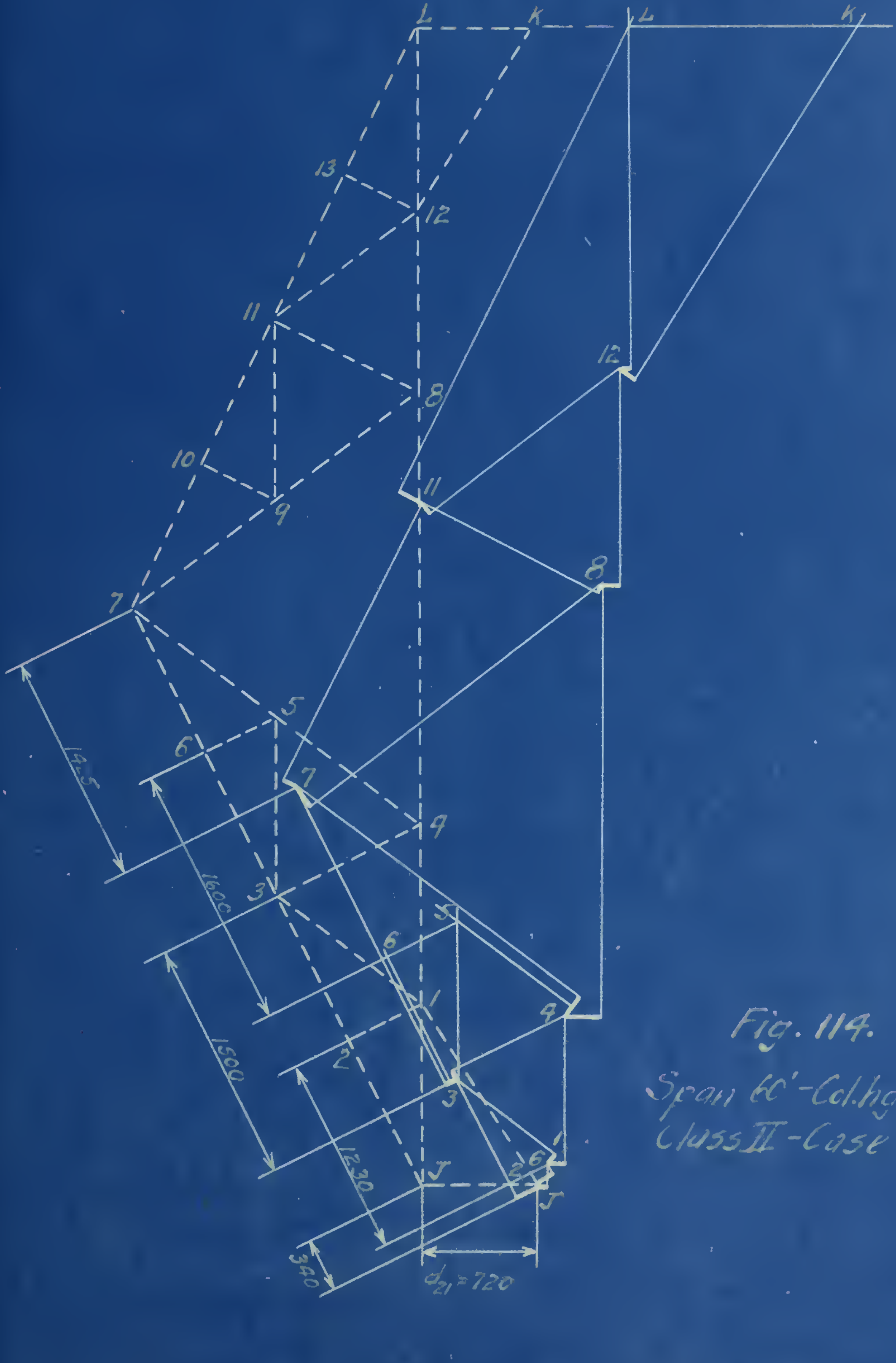


Fig. 114.
Span 60' - Col. hgt. 21'.
Class II - Case A.

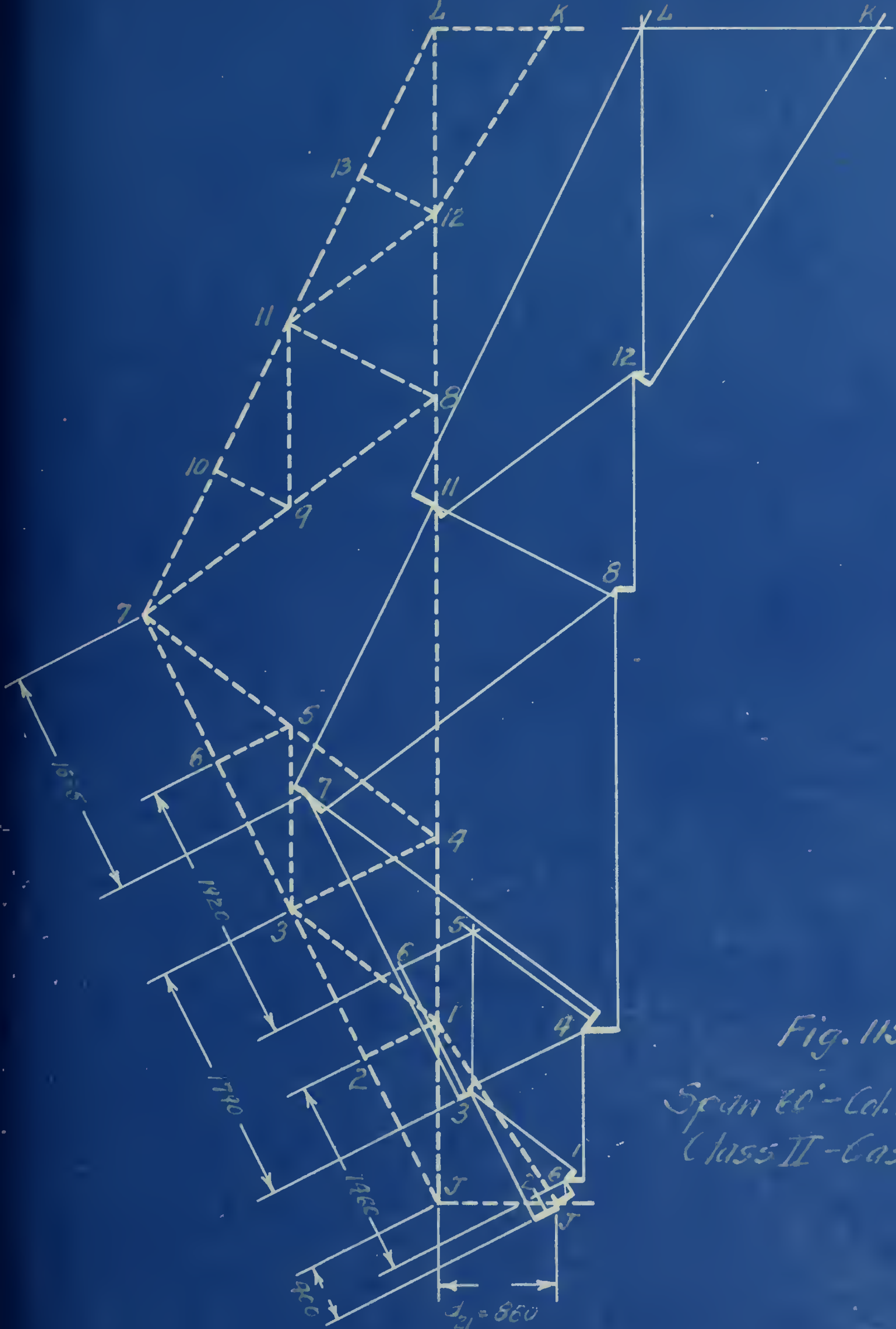


Fig. 115.
Span 60' - Col. hgt. 26'.
Class II - Case A.

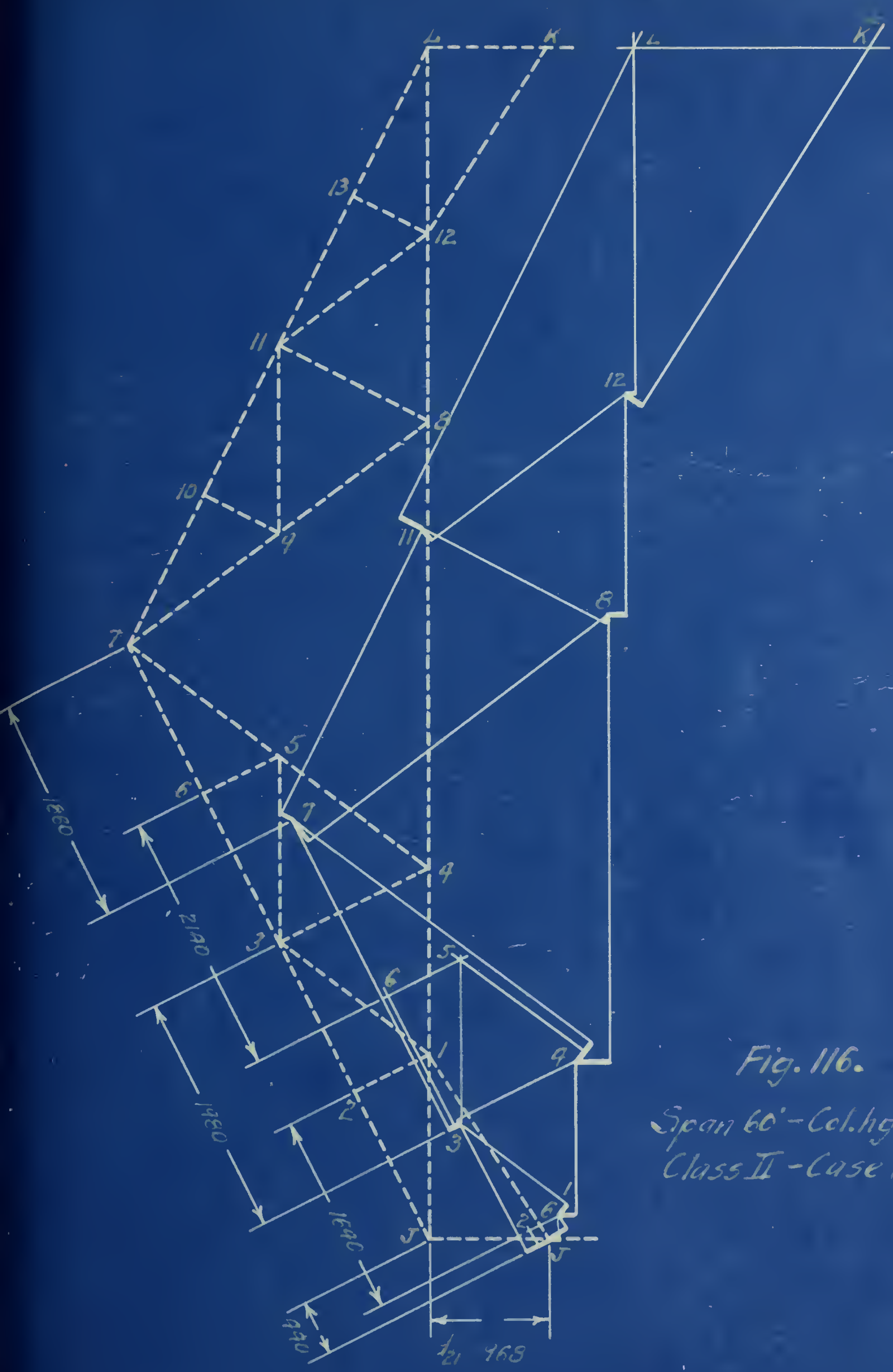


Fig. 116.

Span 60' - Col. hgt. 31.'
Class II - Case A.

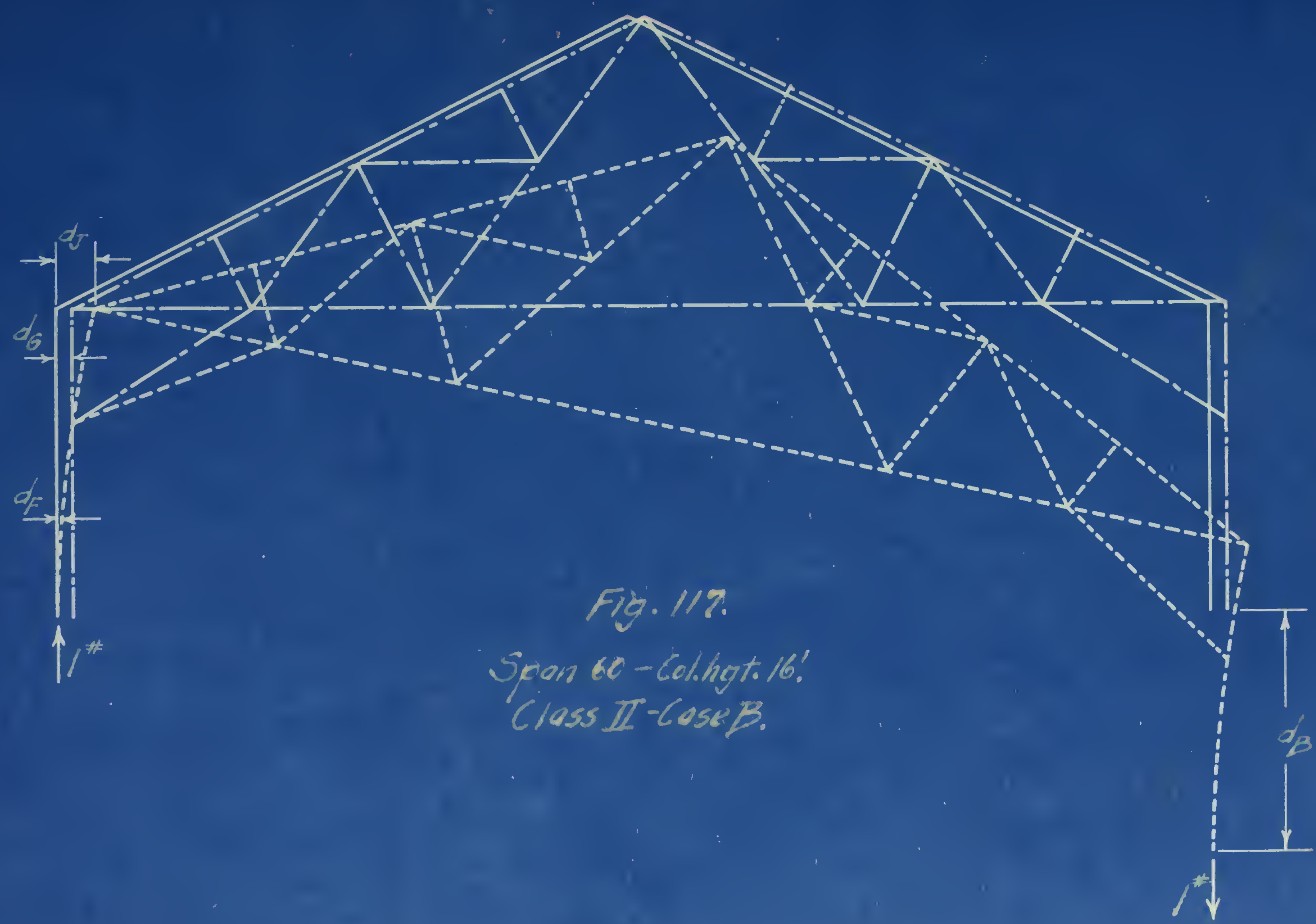


Fig. 117.

Span 60 - Col. hgt. 16'

Class II - Case B.

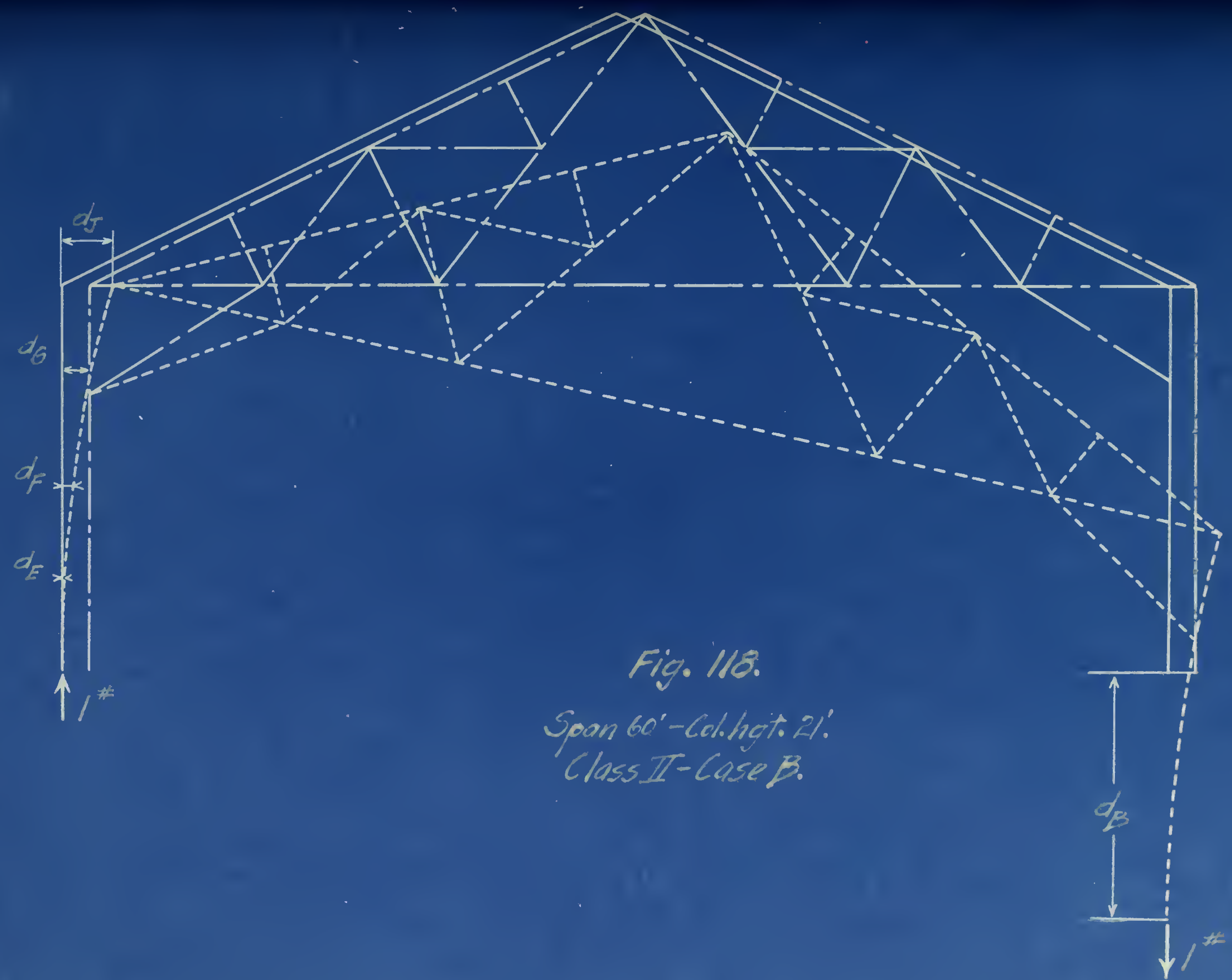


Fig. 118.

Span 60'-Col. hgt. 21'.
Class II-Case B.

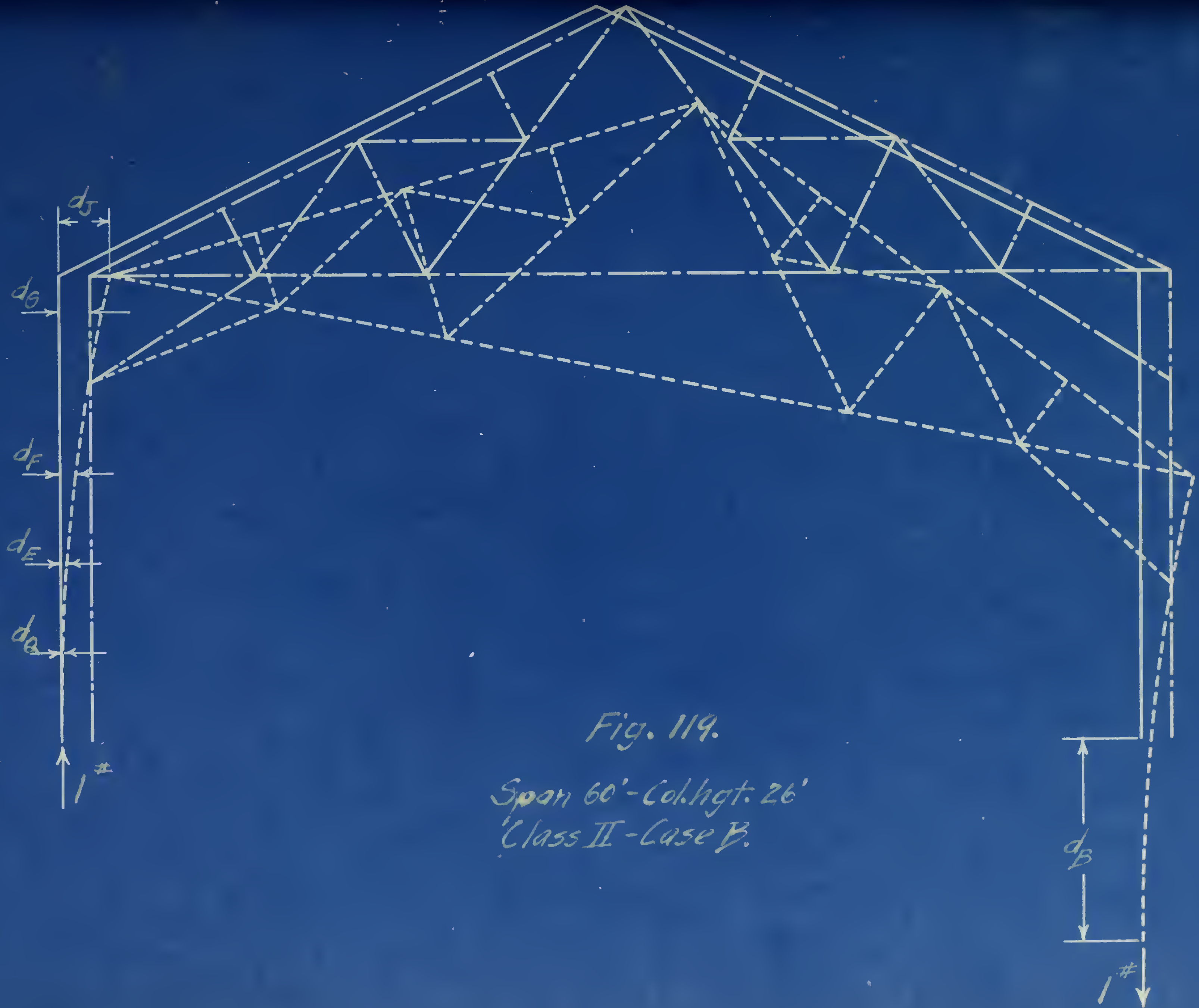


Fig. 119.

Span 60'-Col. hgt. 26'
Class II-Case B.

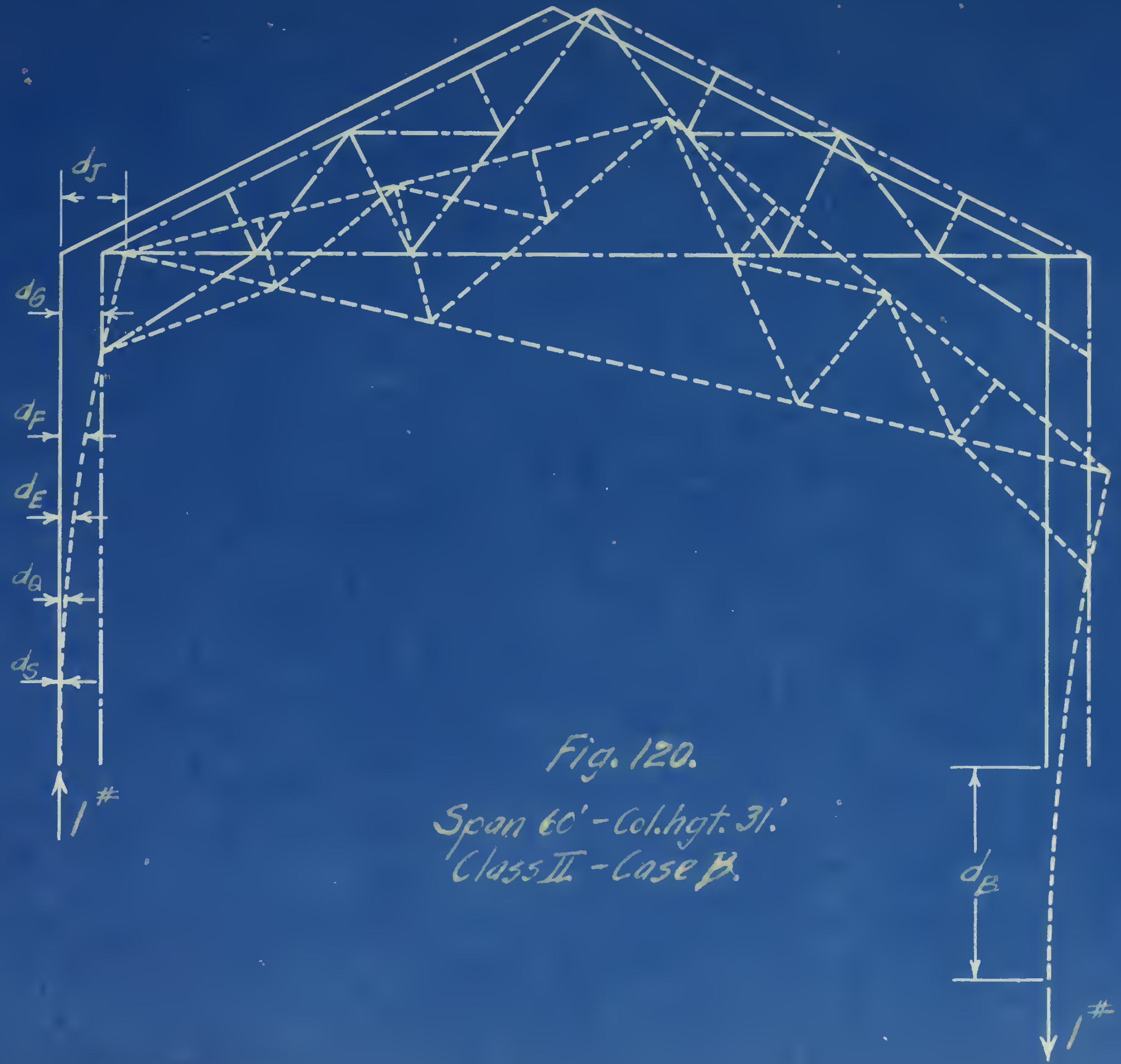


Fig. 120.

Span 60' - Col. hgt. 31'.
Class II - Case B.

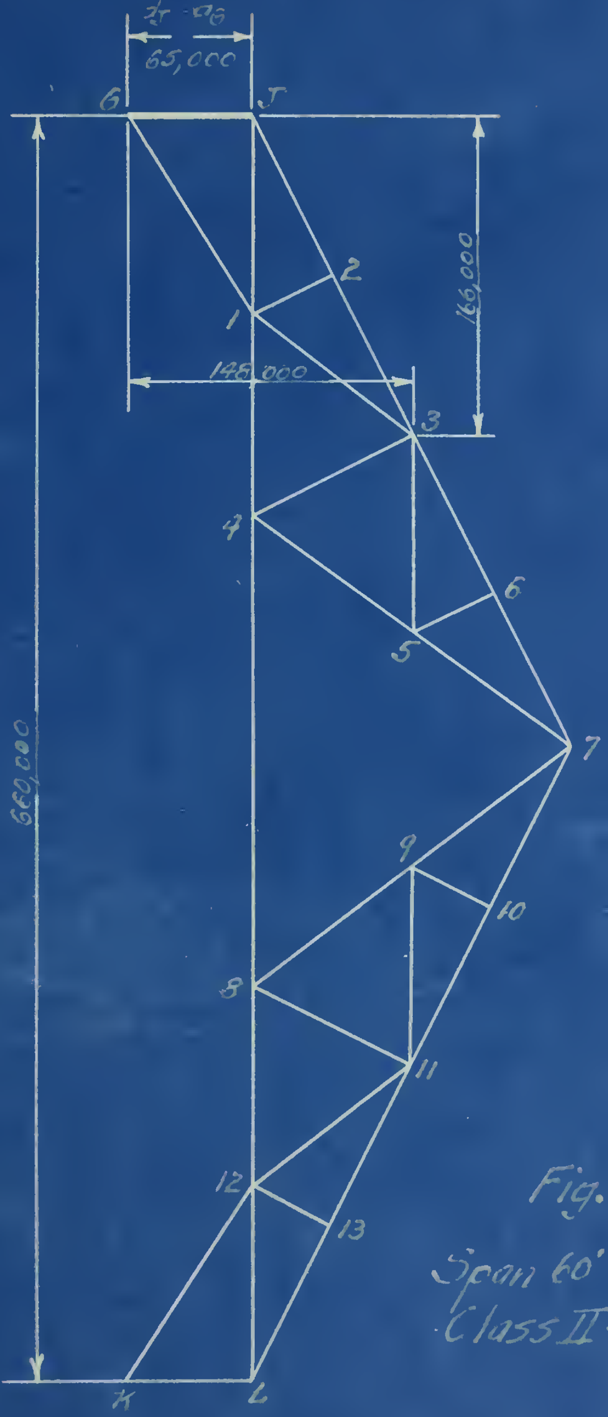


Fig. 121.

Span 60' - Col. hgt. 21.
Class II - Case B.

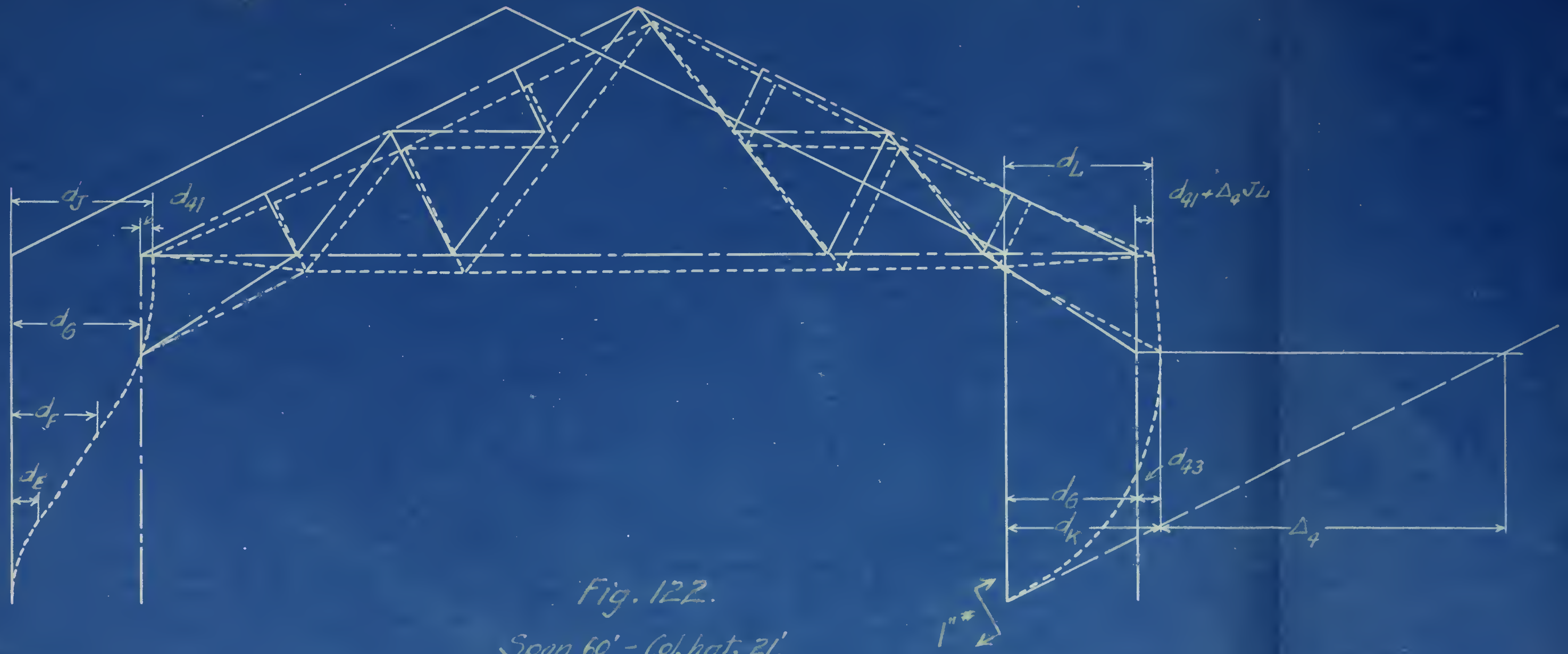


Fig. 122.

Span 60' - Col. hgt. 21'.
Class II - Case C.

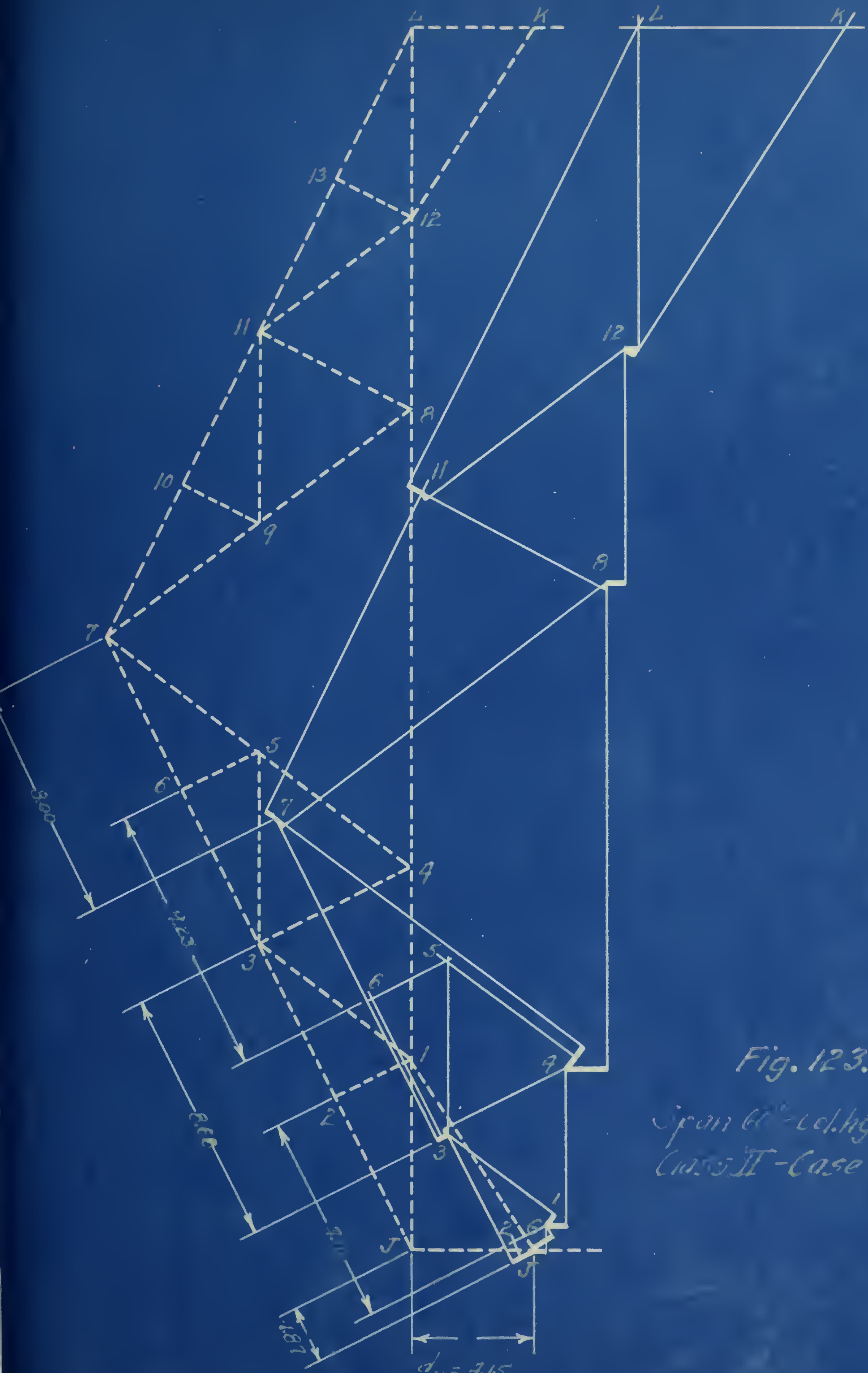


Fig. 123.

Span 60' - col. hgt. 16'
Class II - Case C.

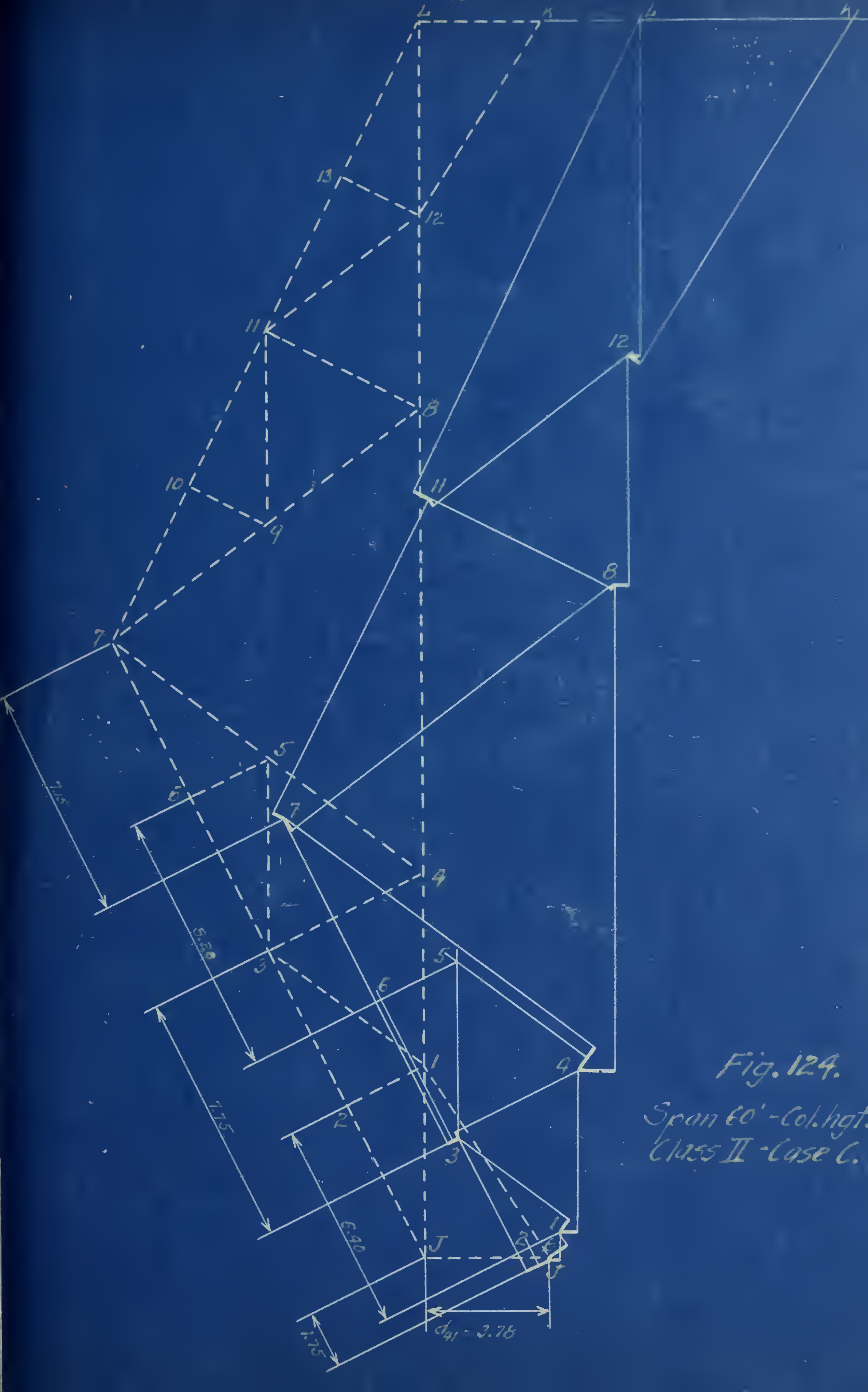


Fig. 124.

Span 60' - Col. hgt. 21
Class II - Case C.

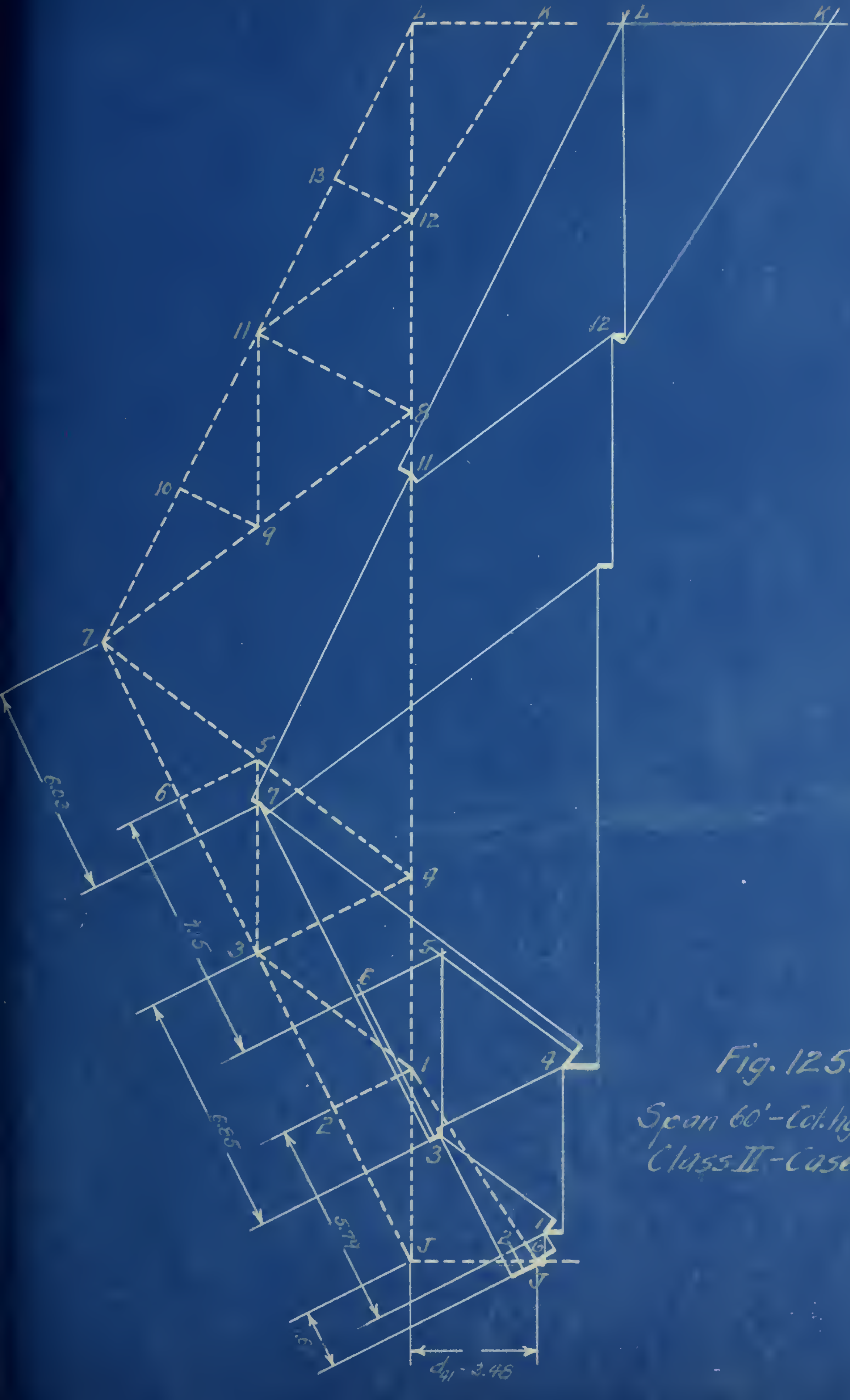


Fig. 125.
Span 60' - Col. hgt. 21'
Class II - Case C.

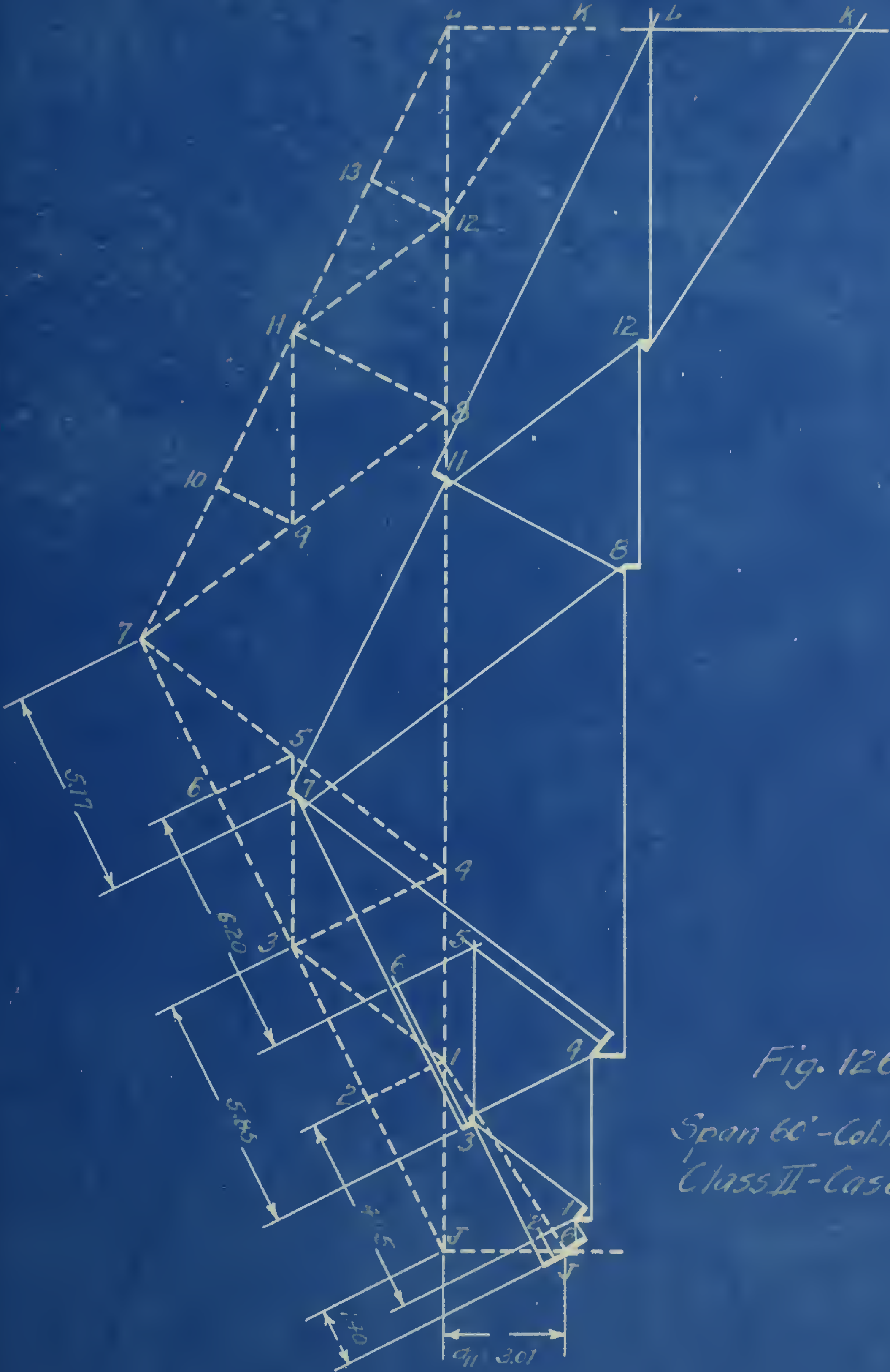


Fig. 126.

Span 60' - Col. hgt. 31'.
Class II - Case C.

TABLE VI

SPAN LENGTH 40 FT.

Member	Section	A Area in sq. in.	l Length in ins	$\frac{l}{A}$
G J	Same as column	7.24	72.00	9.9448
J 1	2 Ls $2\frac{1}{4}$ x $2\frac{1}{4}$ x $\frac{1}{4}$	2.12	75.00	35.3773
G 1		2.12	103.97	49.0424
J 2	2 Ls 2 3/4 x 2 3/4 x $\frac{1}{4}$	2.62	67.08	25.6031
1 2	1 L $2\frac{1}{4}$ x $2\frac{1}{4}$ x $\frac{1}{4}$	1.06	33.54	31.6415
2 3		2.62	67.08	25.6031
1 3		2.12	75.00	35.3773
1 4		2.12	75.00	35.3773
3 4		2.12	67.08	31.6415
3 5	All members having	1.06	75.00	70.7547
4 5		1.06	75.00	70.7547
3 6	the same area have	2.62	67.08	25.6031
5 6		1.06	33.54	31.6415
6 7	the same section	2.62	67.08	25.6031
5 7		1.06	75.00	70.7547
4 8		2.12	180.00	84.9057
7 9		1.06	75.00	70.7547
7 10		2.62	67.08	25.6031
9 10		1.06	33.54	31.6415
10 11		2.62	67.08	25.6031
8 9		1.06	75.00	70.7547
9 11		1.06	75.00	70.7547
8 11		2.12	67.08	31.6415
8 12		2.12	75.00	35.3773
11 12		2.12	75.00	35.3773
11 13		2.62	67.08	25.6031
12 13		1.06	33.54	31.6415
13 L		2.62	67.08	25.6031
12 K		2.12	103.97	49.0424
12 L		2.12	75.00	35.3773
K L		7.24	72.00	9.9448

The Columns D J and B L are made of 1 Pl. 8 x $\frac{1}{4}$ and 4 Ls 3 x $2\frac{1}{2}$ x $\frac{1}{4}$ with the longer leg outstanding. The moment of inertia of the column section about a vertical axis normal to the plane of the truss is 81.2 inches.^{4}

3   222

Member		$\frac{P_2 U_1 l}{A}$		$\frac{P_2 l}{A}$
		A	#	
G	J	+7.7331	+7.7331	-19.05
J	1	+20.1792	+42.1131	+100.19
G	1	+94.2375	+94.2375	+135.36
J	2	+99.5428	+99.5428	-109.58
2	3	+99.5428	+99.5428	-109.58
1	3	+50.9434	+50.9434	+84.91
1	4	+38.2075	+70.0471	+120.28
3	4	+9.1142	+9.1142	-33.54
4	5	+25.4717	+25.4717	+84.91
3	6	+20.2773	+20.2773	-54.79
6	7	+20.2773	+20.2733	-54.79
5	7	+25.4717	+25.4717	+84.91
4	8	+15.2830	+40.7547	+186.79
7	9			+84.91
7	10	+9.2167	+9.2167	-54.79
10	11	+9.2167	+9.2167	-54.79
8	9			+84.91
8	11			-33.54
88	12	+12.7358	+23.3480	+120.28
11	12			+84.91
11	13	+18.4334	+18.4334	-109.58
13	L	+18.4334	+18.4334	-109.58
12	K			+135.36
12	L	+9.7641	+20.3773	+100.19
K	L	+1.4321	+1.4321	-19.05
		+605.5137	+705.9853	

.3581 inches

705.9853 inches

TABLE II.

SPAN 40' - COL. HGT. 21'.

Member	P_1	$\frac{P_1 l}{A}$	U_1	$\frac{P_1 U_1 l}{A}$	P_2		$\frac{P_2 l}{A}$		$\frac{P_2 U_1 l}{A}$		$\frac{P_2 l}{A}$
					A	#	A	#	A	#	
G J	-3.370	-33.514	-0.8100	+27.415	-0.9600	-0.9600	-9.5470	-9.5470	+7.7331	+7.7331	-19.05
J 1	+4.230	+149.646	+0.6200	+93.529	+0.9200	+1.9200	+32.5471	+67.9244	+20.1792	+42.1131	+100.19
G 1	+4.860	+238.346	+1.3862	+332.493	+1.3862	+1.3862	+67.9826	+67.9826	+94.2375	+94.2375	+135.36
J 2	-7.495	-191.895	-1.8112	+349.248	-2.1466	-2.1466	-54.9596	-54.9596	+99.5428	+99.5428	-109.58
2 3	-7.495	-191.895	-1.8112	+349.248	-2.1466	-2.1466	-54.9596	-54.9596	+99.5428	+99.5428	-109.58
1 3	+4.220	+149.292	+1.2000	+180.793	+1.2000	+1.2000	+42.4528	+42.4528	+50.9434	+50.9434	+84.91
1 4	+5.195	+183.785	+0.9000	+164.488	+1.2000	+2.2000	+42.4528	+77.8301	+38.2075	+70.0471	+120.28
3 4	-1.875	- 59.328	-0.5367	+31.859	-0.5367	-0.5367	-16.9820	-16.9820	+9.1142	+9.1142	-33.54
4 5	+2.100	+148.585	+0.6000	+89.151	+0.6000	+0.6000	+42.4528	+42.4528	+25.4717	+25.4717	+84.91
3 6	-3.745	-95.884	-0.7379	+70.187	-1.0733	-1.0733	-27.4798	-27.4798	+20.2773	+20.2773	-54.79
6 7	-3.745	-95.884	-0.7379	+70.187	-1.0733	-1.0733	-27.4798	-27.4798	+20.2773	+20.2733	-54.79
5 7	+2.100	+148.585	+0.6000	+89.151	+0.6000	+0.6000	+42.4528	+42.4528	+25.4717	+25.4717	+84.91
4 8	+3.100	+263.208	+0.3000	+76.857	+0.6000	+1.6000	+50.9434	+135.8491	+15.2830	+40.7547	+186.79
7 9	+2.100	+148.585			+0.6000	+0.6000	+42.4528	+42.4528			+84.91
7 10	-3.745	-95.884	-0.3354	+31.642	-1.0733	-1.0733	-27.4798	-27.4798	+9.2167	+9.2167	-54.79
10 11	-3.475	-95.884	-0.3354	+31.642	-1.0733	-1.0733	-27.4798	-27.4798	+9.2167	+9.2167	-54.79
8 9	+2.100	+148.585			+0.6000	+0.6000	+42.4528	+42.4528			+84.91
8 11	-1.875	-59.328			-0.5367	-0.5367	-16.9820	-16.9820			-33.54
88 12	+5.195	+183.785	+0.3000	+53.665	+1.2000	+2.2000	+42.4528	+77.8301	+12.7358	+23.3490	+120.28
11 12	+4.220	+149.292			+1.2000	+1.2000	+42.4528	+42.4528			+84.91
11 13	-7.495	-191.895	-0.3354	+63.325	-2.1466	-2.1466	-54.9596	+54.9596	+18.4334	+18.4334	-109.58
13 L	-7.495	-191.895	-0.3354	+63.325	-2.1466	-2.1466	-54.9596	-54.9596	+18.4334	+18.4334	-109.58
12 K	+4.860	+238.346			+1.3862	+1.3862	+67.9826	+67.9826			+135.36
12 L	+4.230	+149.646	+0.3000	+43.697	+0.9200	+1.9200	+32.5471	+67.9244	+9.7641	+20.3773	+100.19
K L	-3.370	-33.514	-0.1[illegible]00	+ 5.027	-0.9600	-0.9600	- 9.5470	-9.5470	+1.4321	+1.4321	-19.05
				+2216.927					+605.5137	+705.9853	

$\Delta_{JL} = \frac{930.070}{E}$ inches

$\sum \frac{P_1 U_1 l}{A\,E} = \frac{2216.927}{E}$ inches.

$\Delta_{JL} = \frac{200.9432A + 427.3581}{E}$ inches

$\sum \frac{P_2 U_1 l}{AE} = \frac{605.5137A + 705.9853}{E}$ inches

5	$\frac{P_3 U_5 l}{A}$	$\frac{P_4 U_1 l}{A}$ A	$\frac{P_4 U_1 l}{A}$ E	$\frac{P_4 U_1 l}{A}$ R	$\frac{P_4 l}{A}$
.4000	+118.141			8.0553	-.0910
.1333	+155.970			43.8679	+.4606
.2413	+2094.162				+.7131
.0748	+1520.797			103.6920	-.5238
.0748	+1520.797	Same as	Same as	103.6920	-.5238
.0000	+1132.072				+.4454
.0000	+424.530	A of $\frac{P_2U_1l}{A}$	# of $\frac{P_2U_1l}{A}$	63.6791	+.5646
.5777					-.1782
.0000	+566.036				+.4454
.9194	+168.972			42.2450	-.2358
.9194	+168.972			42.2450	-.2358
.0000	+566.036				+.4454
.0000				50.9434	+.8206
					+.1850
.2361	-76.806			42.2450	-.1724
.2361	-76.806			42.2450	-.1724
					+.1850
					-.0741
.0000	-141.510			63.6791	+.4344
					+.1850
.2361	-281.632			103.6920	-.2921
.2361	-281.632			103.6920	-.2921
					+.2961
.0000	-75.470			43.8679	+.3912
.0000	-21.878			8.0553	-.0507
	+7480.751	+605.5137	705.9853	865.8960	

510 inches

$$\sum \frac{P_4U_1l}{A\,E} = \frac{605.5137A + 705.9853C + 865.8960R}{E} \text{ in.}$$

TABLE III.

SPAN 40' - COL. HGT. 21'.

Member	P_3 A	P_3 C	P_3 #	$\frac{P_3 L}{A}$ A	$\frac{P_3 L}{A}$ C	$\frac{P_3 L}{A}$ #	U_3	$\frac{P_3 U_3 l}{A}$ A	$\frac{P_3 U_3 l}{A}$ C	$\frac{P_3 U_3 l}{A}$ #
G J			-1.0000			-9.9448	-0.9600	+9.1651	+9.1651	+9.5470
J 1			+2.0000			+70.7546	+1.9200	+62.4904	+130.4148	+135.8488
G 1							+1.3862	+94.2375	+94.2375	
J 2			+2.2361			-57.2511	-2.1466	+117.9763	+117.9763	+122.8952
2 3	Same as	Same as	-2.2361	Same as	Same as	-57.2511	-2.1466	+117.9763	+117.9763	+122.8952
1 3							+1.2000	+50.9434	+50.9434	
1 4	A of P_2	# of P_2	+2.0000	A of $P_2 l/A$	# of $P_2 l/A$	+80.7546	+2.2000	+93.3962	+171.2262	+155.6601
3 4							-0.5367	+9.1142	+9.1142	
4 5							+0.6000	+25.4717	+25.4717	
3 6			-2.2361			-57.2511	-1.0733	+29.4941	+29.4941	+61.4476
6 7			-2.2361			-57.2511	-1.0733	+29.4941	+29.4941	+61.4476
5 7							+0.6000	+25.4717	+25.4717	
4 8			+2.0000			+169.8114	+1.6000	+81.5094	+217.3586	+271.6982
7 9			+4.0000			+283.0188	+0.6000	+25.4717	+25.4717	+169.8113
7 10			-4.9194			+125.9519	-1.0733	+29.4941	+29.4941	+135.1842
10 11			-4.9194			-125.9519	-1.0733	+29.4941	+29.4941	+135.1842
6 9			+4.0000			+283.0188	+0.6000	+25.4717	+25.4717	+169.8113
8 11			-3.5777			-113.2038	-0.5367	+9.1142	+9.1142	+60.7565
8 12			+6.0000			+212.2638	+2.2000	+93.3962	+171.2262	+466.9804
11 12			+8.0000			+283.0184	+1.2000	+50.9434	+50.9434	+339.6221
11 13			-12.0748			-309.1523	-2.1466	+117.9763	+117.9763	+663.6263
13 L			-12.0748			-309.1523	-2.1466	+117.9763	+117.9763	+663.6263
12 K			+9.2413			+453.2155	+1.3862	+94.2375	+94.2375	+628.2473
12 L			+4.1333			+146.2250	+1.9200	+62.4904	+130.4148	+280.7520
I L			-5.4000			-53.7019	-0.9600	+9.1651	+9.1651	+51.5538
								+1411.9714	+1839.3294	+4706.5954

Member	$\frac{P_3 l}{A}$	U_5	$\frac{P_3 U_5 l}{A}$	$\frac{P_4 U_1 l}{A}$ A	$\frac{P_4 U_1 l}{A}$ E	$\frac{P_4 U_1 l}{A}$ R	$\frac{P_4 l}{A}$
G J	+21.878	+5.4000	+118.141			8.0553	-.0910
J 1	-37.735	-4.1333	+155.970			43.8679	+.4606
G 1	-226.609	-9.2413	+2094.162				+.7131
J 2	+125.948	+12.0748	+1520.797			103.6920	-.5238
2 3	+125.948	+12.0748	+1520.797	Same as	Same as	103.6920	-.5238
1 3	-141.509	-8.0000	+1132.072				+.4454
1 4	-70.755	-6.0000	+424.530	A of $\frac{P_2 U_1 l}{A}$	# of $\frac{P_2 U_1 l}{A}$	63.6791	+.5646
3 4	+56.607	+3.5777					-.1782
4 5	-141.509	-4.0000	+566.036				+.4454
3 6	+34.348	+4.9194	+168.972			42.2450	-.2358
6 7	+34.348	+4.9194	+168.972			42.2450	-.2358
5 7	-141.509	-4.0000	+566.036				+.4454
4 8	0	-2.0000				50.9434	+.8206
7 9	+141.509						+.1850
7 10	-34.348	+2.2361	-76.806			42.2450	-.1724
10 11	-34.348	+2.2361	-76.806			42.2450	-.1724
6 9	+141.509						+.1850
8 11	-56.607						-.0741
8 12	+70.755	-2.0000	-141.510			63.6791	+.4344
11 12	+141.509						+.1850
11 13	-125.948	+2.2361	-281.632			103.6920	-.2921
13 L	-125.948	+2.2361	-281.632			103.6920	-.2921
12 K	+226.609						+.2961
12 L	+37.735	-2.0000	-75.470			43.8679	+.3912
I L	-21.878	+1.0000	-21.878			8.0553	-.0507
			+7480.751	+605.5137	705.9853	865.8960	

$\Delta JL = \frac{200.9432A + 427.3581C + 670.0936}{E}$ inches

$\sum \frac{P_3 U_3 l}{AE} = \frac{1411.9714A + 1839.3294C + 4706.5954}{E}$ in.

$\sum \frac{P_3 U_5 l}{AE} = \frac{7480.7510}{E}$ inches

$\sum \frac{P_4 U_1 l}{AE} = \frac{605.5137A + 705.9853C + 865.8960R}{E}$ in.

TABLE IV.

SPAN 40' - COL. HGT. 21'.

Class I

Load	Vert. Reacts. (Lbs.)		Horiz. Reacts. (Lbs)		Rest. Moms. (In. Lbs.)	
	L. C.	R. C.	L. C.	R. C.	L. C.	R. C.
1# at E	-0.1250	+0.1250	+0.8016	+0.1984	0	0
1# at F	-0.2500	+0.2500	+0.6350	+0.3650	0	0
1# at G	-0.3750	+0.3750	+0.5321	+0.4679	0	0
1# at J	-0.5250	+0.5250	+0.5056	+0.4944	0	0
1# at 3	+0.3805	+0.5137	+0.1968	+0.2504	0	0

Class II

Load	Vert. Reacts. (Lbs.)		Horiz. Reacts. (Lbs.)		Rest. Moms. (In. Lbs.)	
	L. C.	R. C.	L. C.	R. C.	L. C.	R. C.
1# at E	-0.0179	+0.0179	+0.9129	+0.0871	+42.090	+19.318
1# at F	-0.0717	+0.0717	+0.7308	+0.2692	+56.873	+28.711
1# at G	-0.1613	+0.1613	+0.5728	+0.4272	+57.239	+45.337
1# at J	-0.3075	+0.3075	+0.5141	+0.4859	+53.391	+51.009
1# at 3	+0.4799	+0.4143	+0.1648	+0.2824	+18.826	+28.910

Upward, to the left and counterclockwise, positive.

TABLE V.

SPAN LENGTH 20 FT

Member	Section	A Area in sq. in.	l Length in inches	$\frac{l}{A}$
GJ	1 Pl 10x1/4 4 L^s 3 1/2x2 1/2x1/4	8.26	48.0	5.811138014528
GT	2L^s 2 1/2x2x1/4	2.12	89.044932477935	42.002326640535
JT	2L^s 2 1/2x2x1/4	2.38	75.0	31.512605042017
JO	2L^s 3x2 1/2x1/4	2.62	67.082039324993	25.603831803430
OT	1L 2 1/2x2x1/4	1.06	33.541019762497	31.642471474054
OS	2L^s 3x2 1/2x1/4	2.62	67.082039324993	25.603831803430
TS	1L 2 1/2x2x1/4	1.06	75.0	70.754716981132
TU	2L^s 2 1/2x2 1/2x1/4	2.38	90.0	37.815126050420
SU	1L 2 1/2x2x1/4	1.06	75.0	70.754716981132
SL	2L^s 3x2 1/2x1/4	2.62	134.164078649987	51.207663606860
UL	2L^s 2 1/2x2 1/2x1/4	2.38	75.0	31.512605042017
UK	2L^s 2 1/2x2x1/4	2.12	89.044932477935	42.002326640535
KL	1 Pl 10x1/4 4L^s 3 1/2x2 1/2x1/4	8.26	48.0	5.811138014528

The columns DJ and BL are made of 1 Pl 10x1/4 and 4L^s 3 1/2x2 1/2x1/4, 3 1/2 leg outstanding.

The moment of inertia of the column section about a neutral axis normal to the plane of the truss is 135 inches^4.

TABLE VI.

SPAN LENGTH 30 FT.

Member	Section	A Area sq. in.	L Length in.	$\frac{L}{A}$
G J	Same as columns.	7.24	48,000	6.6298
J 1	2 LS L2½ x 2 x ¼	2.12	56,250	26.5330
G 1		2.12	73,946	34.8802
J 2	2 LS 2½ x 2½ x ¼	2.38	50.312	21.1395
1 2	1 L 2½ x 2 x ¼	1.06	25.156	23.7321
2 3		2.38	50.312	21.1395
1 3		2.12	56.250	26.5330
1 4		2.12	56.250	26.5330
3 4	All members having	2.12	50.312	23.7321
3 5		1.06	56.250	53.0660
4 5	The same area have	1.06	56.250	53.0660
3 6		2.38	50.312	21.1395
5 6	The same section	1.06	25.156	23.7321
6 7		2.38	50.312	21.1395
5 7		1.06	56.250	53.0660
4 8		2.12	135.000	63.6792
7 9				
7 10				
9 10				
10 11				
8 9				
9 11				
8 11	Symmetrical with the other half of Truss.			
8 12				
11 12				
11 13				
12 13				
13 L				
12 K				
12 L				
K L				

The columns DJ and BL are made of 1 Pl. 8 x ¼ and 4 LS 3 x 2½ x ¼ with the longer leg outstanding. The moment of inertia of the column section about a vertical axis normal to the plane of the truss is 81.2 inches.^{4}

TABLE VII.

SPAN LENGTH 40 FT.

Member	Section	A Area in sq. in.	l Length in ins	$\frac{l}{A}$
G J	Same as column	7.24	72.00	9.9448
J 1	2 Ls $2\frac{1}{4}$ x $2\frac{1}{4}$ x $\frac{1}{4}$	2.12	75.00	35.3773
G 1		2.12	103.97	49.0424
J 2	2 Ls 2 3/4 x 2 3/4 x $\frac{1}{4}$	2.62	67.08	25.6031
1 2	1 L $2\frac{1}{4}$ x $2\frac{1}{4}$ x $\frac{1}{4}$	1.06	33.54	31.6415
2 3		2.62	67.08	25.6031
1 3		2.12	75.00	35.3773
1 4		2.12	75.00	35.3773
3 4		2.12	67.08	31.6415
3 5	All members having	1.06	75.00	70.7547
4 5		1.06	75.00	70.7547
3 6	the same area have	2.62	67.08	25.6031
5 6		1.06	33.54	31.6415
6 7	the same section	2.62	67.08	25.6031
5 7		1.06	75.00	70.7547
4 8		2.12	180.00	84.9057
7 9		1.06	75.00	70.7547
7 10		2.62	67.08	25.6031
9 10		1.06	33.54	31.6415
10 11		2.62	67.08	25.6031
8 9		1.06	75.00	70.7547
9 11		1.06	75.00	70.7547
8 11		2.12	67.08	31.6415
8 12		2.12	75.00	35.3773
11 12		2.12	75.00	35.3773
11 13		2.62	67.08	25.6031
12 13		1.06	33.54	31.6415
13 L		2.62	67.08	25.6031
12 K		2.12	103.97	49.0424
12 L		2.12	75.00	35.3773
K L		7.24	72.00	9.9448

The Columns D J and B L are made of 1 Pl. 8 x $\frac{1}{4}$ and 4 Ls 3 x $2\frac{1}{2}$ x $\frac{1}{4}$ with the longer leg outstanding. The moment of inertia of the column section about a vertical axis normal to the plane of the truss is81.2 inches.^{4}

TABLE VIII.

SPAN LENGTH 50 FT.

Member.		Section.	A Area sq. in.	L Length in in.	$\frac{L}{A}$
G	5	Same as columns.	7.24	72,000	9.9448
J	1	2 Ls $2\frac{1}{2}$ x $2\frac{1}{2}$ x $\frac{1}{4}$	2.38	93,750	39.3908
G	1		2.38	118.208	49.6671
J	2	2 Ls 3 x 3 x 5/16	3.56	83.853	23.5541
1	2	1 L $2\frac{1}{2}$ x 2 x $\frac{1}{4}$	1.06	41.926	39.5531
2	3		3.56	83.853	23.5541
1	3	2 Ls $2\frac{1}{2}$ x 2 x $\frac{1}{4}$	2.12	93.750	44.2217
1	4		2.38	93.750	39.3908
3	4		2.12	83.853	39.5531
3	5	All members having	1.06	93.750	88.4434
4	5		1.06	93.750	88.4434
3	6	The same area have	3.56	83.853	23.5541
5	6		1.06	41.926	39.5531
6	7	The same section	3.56	83.853	23.5541
5	7		1.06	93.750	88.4434
4	8		2.12	225.000	106.1321
7	9				
7	10				
9	10				
10	11				
8	9				
9	11	Symmetrical with the other half of the Truss.			
8	11				
8	12				
11	12				
11	13				
12	13				
13	L				
12	K				
12	L				
K	L				

The columns D N and B L are made of 1 Pl. 8 x $\frac{1}{4}$ and 4 Ls 3 x $2\frac{1}{2}$ x $\frac{1}{4}$ with the longer leg outstanding. The moment of inertia of the column section about a vertical axis normal to the plane of the truss is 81.2 inches.^{4}

TABLE IX.

SPAN LENGTH 60 FT.

Member	Section	A Area in sq. in	l Length in ins	$\frac{l}{A}$
G J	Same as column	7.24	72.00	4.9448
J 1	2 ⌊s 2 3/4 x 2 3/4 x ¼	2.62	112.50	42.9389
G 1		2.62	133.57	50.9809
J 2	2 ⌊s 3½ x 3 x 5/16	3.86	100.62	26.0674
1 2	1 ⌊ 2¼ x 2¼ x ¼	1.06	50.31	47.4623
2 3		3.86	100.62	26.0674
1 3	2 ⌊s 2¼ x 2¼ x ¼	2.12	112.50	53.0660
1 4		2.62	112.50	42.9389
3 4		2.12	100.62	47.4623
3 5		1.06	112.50	106.1321
4 5	1 ⌊ 2½ x 2½ x ¼	1.19	112.50	94.5378
3 6		3.86	100.62	26.0674
5 6		1.06	50.31	47.4623
6 7		3.86	100.62	26.0674
5 7	All members having	1.19	112.50	94.5378
4 8		2.12	270.00	127.3585
7 9	the same area have	1.19	112.50	94.5378
7 10		3.86	100.62	26.0674
9 10	the same section	1.06	50.31	47.4623
10 11		3.86	100.62	26.0674
8 9		1.19	112.50	94.5378
9 11		1.06	112.50	106.1321
8 11		2.12	100.62	47.4623
8 12		2.62	112.50	42.9389
11 12		2.12	112.50	53.0660
11 13		3.86	100.62	26.0674
12 13		1.06	50.31	47.4623
13 L		3.86	100.62	26.0674
12 K		2.62	133.57	50.9809
12 L		2.62	112.50	42.9389
K L		7.24	72.00	9.9448

The columns D J and B L are made of 1 Pl. 8x 1/4 and 4 ⌊s 3 x 2½ x ¼ with longer leg outstanding. The moment of inertia of the column section about a vertical axis normal to the plane of the truss is 81.2 inches^4.

TABLE X.

SPAN 20' – COL. HGT. 12'.

Member.	P_1	$\frac{P_1 L}{A}$	U_1	$\frac{P_1 U_1 L}{A}$	P_2	
					A	#
J	-1.9200	- 11.1574	-0.4400	+ 4.9092	-0.6400	-0.6400
T	+3.5618	+149.6038	+1.1873	+177.6194	+1.1873	+1.1873
T	+1.8400	+ 57.9832	-0.1200	- 6.9580	+0.2800	+1.2800
O	-4.2933	-109.9237	-0.9839	+108.1506	-1.4311	-1.4311
S	-4.2933	-109.9237	-0.9839	+108.1506	-1.4311	-1.4311
S	+2.4000	+169.8113	+0.8000	+135.8491	+0.8000	+0.8000
U	+3.4000	+128.5714	+0.4000	+ 51.4286	+0.8000	+1.8000
U	+2.4000	+169.8113	0	0	+0.8000	+0.8000
Q	-4.2933	-109.9327	-0.4472	+ 49.1594	-1.4311	-1.4311
L	-4.2933	-109.9237	-0.4472	+ 49.1594	-1.4311	-1.4311
L	+1.8400	+ 57.9832	+0.4000	+ 23.1933	+0.2800	+1.2800
K	+3.5618	+149.6038	0	0	+1.1873	-1.1873
K	-1.9200	- 11.1574	-0.2000	+ 2.2315	-0.6400	-0.6400
				+702.8930		

$\Delta JL = \frac{244.5378}{E}$ $\qquad \sum \frac{P_1 U_1 L}{AE} = \frac{702.8930}{E}$

$\frac{P_2 L}{A}$		$\frac{P_2 U_1 L}{A}$		$\frac{P_2 L}{A}$
A	#	A	#	
- 3.7191	- 3.7191	+ 1.6364	+ 1.6364	- 6.1341
+49.8679	+49.8679	+59.2065	+59.2065	+82.2489
+ 8.8235	+40.3361	- 1.0588	- 4.8403	+46.0656
-36.6411	-36.6411	+36.0502	+36.0502	-60.4336
-36.6411	-36.6411	+36.0502	+36.0502	-60.4336
+56.6038	+56.6038	+45.2830	+45.2830	+93.3586
+30.2521	+68.0672	+12.1008	+27.2269	+87.7110
+56.6038	+56.6038	0	0	+93.3586
-36.6411	-36.6411	+16.3865	+16.3865	-60.4336
-36.6411	-36.6411	+16.3865	+16.3865	-60.4336
+ 8.8235	+40.3361	+ 3.5294	+16.1345	+46.0656
+49.8679	+49.8679	0	0	+82.2489
- 3.7191	- 3.7191	+ 0.7438	+ 0.7438	- 6.1341
		+226.3145	+250.2640	

$$\Delta JL = \frac{47.8991\,A + 148.7395}{E}$$

$$\sum \frac{P_2 U_1 L}{AE} = \frac{226.3145\,A + 250.2640}{E}$$

TABLE XI.

SPAN 20' - COL. HGT. 12'.

Member	P_3 A	P_3 C	P_3 #	$\frac{P_3 L}{A}$ A	$\frac{P_3 L}{A}$ C	$\frac{P_3 L}{A}$ #	U_3	$\frac{P_3 U_3 L}{A}$ A	$\frac{P_3 U_3 L}{A}$ C	$\frac{P_3 U_3 L}{A}$ #
G J	Same as A of P_2	Same as # of P_2	-1.0000	Same as A of $\frac{P_2 L}{A}$	Same as # of $\frac{P_2 L}{A}$	-5.8111	-0.6400	+ 2.3802	+ 2.3802	+ 3.7191
G T			-0-			-0-	+1.1873	+59.2065	+59.2065	0
J T			+2.0000			+63.0252	+1.2800	+11.2941	+51.6303	+80.6723
J O			-2.2361			-57.2519	-1.4311	+52.4366	+52.4366	+81.9323
O S			-2.2361			-57.2519	-1.4311	+52.4366	+52.4366	+81.9323
T S			-0-			-0-	+0.8000	+45.2830	+45.2830	0
T U			+2.0000			+75.6302	+1.8000	+54.4538	+122.5210	+136.1345
S U			+4.0000			+283.0189	+0.8000	+45.2830	+45.2830	+226.4151
S Q			-4.9193			-125.9542	-1.4311	+52.4366	+52.4366	+180.2510
Q L			-4.9193			-125.9542	-1.4311	+52.4366	+52.4366	+180.2510
U L			-0.6000			- 18.9076	+1.2800	+11.2941	+51.6303	+24.2017
U K			+5.9363			+249.3396	+1.1873	+59.2065	+59.2065	+206.0324
L K			-2.2000			- 12.7845	-0.6400	+ 2.3802	+2.3802	+8.1821
								+500.5281	+649.2676	+1251.3202

$\Delta 3 L \quad \frac{97.8992 A + 148.7345 C + 119.7979}{E}$

$$\sum \frac{P_3 U_3 L}{AE} = \frac{500.5281 A + 649.2676 C + 1251.3202}{E}$$

Member.	$\frac{P_3 L}{A}$	U_5	$\frac{P_3 U_5 L}{A}$
G J	+ 3.4867	+2.2000	+ 7.6707
G T	-124.6698	-5.9363	+740.0810
J T	+ 40.9664	+0.6000	+ 24.5798
J O	+ 34.3511	+4.9193	+168.9853
O S	+ 34.3511	+4.9193	+168.9853
T S	-141.5094	-4.0000	+566.0377
T U	0	-2.0000	0
S U	+141.5094	0	0
S Q	- 34.3511	+2.2361	- 76.8115
Q L	- 34.3511	+2.2361	- 76.8115
U L	- 40.9664	-2.0000	+ 81.9328
U K	+124.6698	0	0
L K	- 3.4867	+1.0000	- 3.4867
			+1601.1629

$$\sum \frac{P_3 U_5 L}{AE} = \frac{1601.1629}{E}$$

$\frac{P_4 U_1 L}{A}$ A	$\frac{P_4 U_1 L}{A}$ C	$\frac{P_4 U_1 L}{A}$ R	$\frac{P_4 L}{A}$
Same. as A of $\frac{P_2 U_1 L}{A}$	Same. as # of $\frac{P_2 U_1 L}{A}$	+ 2.5569	-0.0408
		0	+0.6990
		- 7.5630	+0.2642
		+56.3284	-0.4018
		+56.3284	-0.4018
		0	+0.7934
		+30.2521	+0.5926
		[illegible]	+0.2409
		+56.3284	-0.2677
		+56.3284	-0.2677
		- 7.5630	+0.4241
		0	+0.2122
		+ 2.5569	-0.0272
		+245.5536	

$$\sum \frac{P_4 U_1 L}{AE} = \frac{226.3145 A + 250.2640 C + 245.5536 R}{E}$$

Member.	P_1.		$\frac{P_2 U_1 L}{A}$ A	$\frac{P_2 U_1 L}{A}$ #	$\frac{P_2 L}{A}$
G J	-3.413	-2	- 4.0732	+ 4.0732	- 12.204
J 1	+3.826	+1	+ 8.2504	+ 9.9249	+ 66.982
G 1	+5.258	+1	+ 60.2790	+ 60.2790	+ 98.915
J 2	-7.632	-1	+ 64.9381	+ 64.9381	- 87.009
2 3	-7.632	-1	+ 64.9381	+ 64.9381	- 87.009
1 3	+4.267	+11	+ 30.1906	+ 30.1906	+ 61.055
1 4	+5.267	+13	+ 22.6422	+ 43.8686	+ 87.588
3 4	-1.908	- 4	+ 5.3997	+ 5.3997	- 24.420
4 5	+2.133	+11	+ 15.0924	+ 15.0924	+ 61.055
3 6	-3.816	- 8	+ 13.2276	+ 13.2276	- 43.505
6 7	-3.816	- 8	+ 13.2276	+ 13.2276	- 43.505
5 7	+2.133	+11	+ 15.0924	+ 15.0924	+ 61.055
4 8	+3.133	+19	+ 9.0572	+ 26.0404	+136.938
7 9		+11	-0-	-0-	+ 61.055
7 10		- 8	+ 6.0501	+ 6.0501	- 43.505
10 11		- 8	+ 6.0501	+ 6.0501	- 43.505
8 9		+11	-0-	-0-	+ 61.055
8 11	symmetrical	- 4	-0-	-0-	- 24.420
8 12		+13	+ 7.5484	+ 14.6247	+ 87.588
11 12		+11	-0-	-0-	+ 61.055
11 13		-16	+ 12.1003	+ 12.1003	- 87.009
13 L		-16	+ 12.1003	+ 12.1003	- 87.009
12 K		+18	-0-	-0-	+ 98.915
12 L		+10	+ 5.0008	+ 12.0772	+ 66.982
K L		- 2	+ 0.7541	+ 0.7541	- 12.204
			+3[illegible]6.0126	+440.0494	

$\Delta\, J\, L = \frac{682.0361}{E}$ + 297.8787 inches.

$\sum \frac{P_1 U_1 L}{A E} = \frac{1568.}{E}$ + 440.0494 inches.

TABLE XII

SPAN 30'-COL. HGT. 16'.

Member.	P_1.	$\frac{P_1 L}{A}$	U_1	$\frac{P_1 U_1 L}{A}$	P_2 A	P_2 #	$\frac{P_2 L}{A}$ A	$\frac{P_2 L}{A}$ #	$\frac{P_2 U_1 L}{A}$ A	$\frac{P_2 U_1 L}{A}$ #	$\frac{P_2 L}{A}$
G J	-3.413	-22.6275	-0.7200	+16.2918	-0.8533	-0.8533	-5.6572	-5.6572	-4.0732	+4.0732	-12.204
J 1	+3.826	+101.5153	+0.4400	+44.6667	+0.7067	+1.7067	+18.7509	+45.2839	+8.2504	+9.9249	+66.982
G 1	+5.258	+183.4000	+1.3146	+241.0976	+1.3146	+1.3146	+45.8535	+45.8535	+60.2790	+60.2790	+98.915
J 2	-7.632	-161.3367	-1.6100	+259.7521	-1.9080	-1.9080	-40.3342	-40.3342	+64.9381	+64.9381	-87.009
2 3	-7.632	-161.3367	-1.6100	+259.7521	-1.9080	-1.9080	-40.3342	-40.3342	+64.9381	+64.9381	-87.009
1 3	+4.267	+113.2163	+1.0667	+120.7678	+1.0667	+1.0667	+28.3028	+28.3028	+30.1906	+30.1906	+61.055
1 4	+5.267	+139.7493	+0.8000	+111.7994	+1.0667	+2.0667	+28.3028	+54.8358	+22.6422	+43.8686	+87.588
3 4	-1.908	-45.2808	-0.4770	+21.5989	-0.4770	-0.4770	-11.3202	-11.3202	+5.3997	+5.3997	24.420
4 5	+2.133	+113.1898	+0.5333	+60.3641	+0.5333	+0.5333	+28.3000	+28.3000	+15.0924	+15.0924	+61.055
3 6	-3.816	-80.6683	-0.6559	+52.9103	-0.9540	-0.9540	-20.1671	-20.1671	+13.2276	+13.2276	-43.505
6 7	-3.816	-80.6683	-0.6559	+52.9103	-0.9540	-0.9540	-20.1671	-20.1671	+13.2276	+13.2276	-43.505
5 7	+2.133	+113.1898	+0.5333	+60.3641	+0.5333	+0.5333	+28.3000	+28.3000	+15.0924	+15.0924	+61.055
4 8	+3.133	+199.5069	+0.2667	+53.2085	+0.5333	+1.5333	+33.9601	+97.6393	+9.0572	+26.0404	+136.938
7 9		+113.1898	-0-	-0-			+28.3000	+28.3000	-0-	-0-	+61.055
7 10		-80.6683	-0.3000	+24.2005			-20.1671	-20.1671	+6.0501	+6.0501	-43.505
10 11		-80.6683	-0.3000	+24.2005			÷20.1671	-20.1671	+6.0501	+6.0501	-43.505
8 9	Symmetrical	+113.1898	-0-	-0-			+28.3000	+28.3000	-0-	-0-	+61.055
8 11		-45.2808	-0-	-0-	Symmetrical.		-11.3202	-11.3202	-0-	-0-	-24.420
8 12		+139.7493	+0.2667	+37.2711			+28.3028	+54.8358	+7.5484	+14.6247	+87.588
11 12		+113.2163	-0-	-0-			+28.3028	+28.3028	-0-	-0-	+61.055
11 13		-161.3367	-0.3000	+48.4010			-40.3342	-40.3342	+12.1003	+12.1003	-87.009
13 L		-161.3367	-0.3000	+48.4010			-40.3342	-40.3342	+12.1003	+12.1003	-87.009
12 K		+183.4000	-0-	-0-			+45.8535	+45.8535	-0-	-0-	+98.915
12 L		+101.5153	+0.2667	+27.0741			+18.7509	+45.2839	+5.0008	+12.0772	+66.982
K L		-22.6275	-0.1333	+3.0162			-5.6572	-5.6572	+0.7541	+0.7541	-12.204
				+1568.0481					+376.0126	+440.0494	

$\Delta J L = \frac{682.0361}{E}$ inches.

$\sum \frac{P_1 U_1 L}{A E} = \frac{1568.0481}{E}$ inches.

$\Delta J L = \frac{128.0675 A + 297.8787}{E}$ inches.

$\sum \frac{P_1 U_2 L}{A E} = \frac{376.0126 A + 440.0494}{E}$ inches.

U_5	$\frac{PUL_{35}}{A}$	$\frac{PUL_{41}}{A}$			$\frac{PL_4}{A}$
		A	C	R	
+5.4000	+ 78.754	Same as A of $\frac{PUL_{21}}{A}$	Same as # of $\frac{PUL_{21}}{A}$	+ 4.7729	-0.0724
-3.3000	+ 56.925			+23.3505	+0.3758
-9.8595	+1695.351			-0-	+0.6516
+12.0750	+1252.045			+76.5778	-0.5161
+12.0750	+1252.045			+76.5778	-0.5161
- 8.0000	+ 849.080			-0-	+0.4021
- 6.0000	+ 318.414			+42.4554	+0.5114
+ 3.5777	+ 151.877			-0-	-0.1609
- 4.0000	+ 424.540			-0-	+0.4021
+ 4.9193	+ 138.050			+31.1972	-0.2295
+ 4.9193	+ 138.050			+31.1972	-0.2295
- 4.0000	+ 424.540			-0-	+0.4021
- 2.0000	-0-			+33.9665	+0.7450
-0-	-0-			-0-	+0.1474
+ 2.2500	- 63.821			+31.1972	-0.1618
+ 2.2500	- 63.821			+31.1972	-0.1618
-0-	-0-			-0-	+0.1474
-0-	-0-			-0-	-0.0590
- 2.0000	- 106.126			+42.4554	+0.3841
-0-	-0-			-0-	+0.1474
+ 2.2500	- 234.014			+76.5778	-0.2669
+ 2.2500	- 234.014			+76.5778	-0.2669
-0-	-0-			-0-	+0.2389
[illegible].0000	- 34.486			+23.3505	+0.3344
+ 1.0000	- 14.586			+ 4.7729	-0.0374
	+6028.803			+606.2241	

$$\sum \frac{PUL_{35}}{AE} = \frac{6,028.803}{E}$$

$$\sum \frac{PUL_{41}}{AE} = \frac{+376.0126A + 440.0494C + 606.2241R}{E}$$

TABLE XIII.

SPAN 30'- COL. HGT. 16'.

Member.	P_3 A	P_3 C	P_3 #	$\frac{P_3L}{A}$ A	$\frac{P_3L}{A}$ C	$\frac{P_3L}{A}$ #
G J	Same. as A of P_2	Same. as # of P_2	-1.0000	Same. as A of $\frac{P_2L}{A}$	Same. as # of $\frac{P_2L}{A}$	-6.6298
J I *			+2.0000			+53.0660
G 1			-0-			-0-
J 2			-2.2500			-47.5638
2 3			-2.2500			-47.5638
1 3			-0-			-0-
1 4 *			+2.0000			+53.0660
3 4			-0-			-0-
4 5			-0-			-0-
3 6			-2.2500			-47.5638
6 7			-2.2500			-47.5638
5 7			-0-			-0-
4 8 *			+2.0000			+127.3584
7 9			+4.0000			+212.2640
7 10			-4.9193			-103.9915
10 11			-4.9193			-103.9915
8 9			+4.0000			+212.2640
8 11			-3.5777			-84.9063
8 12 *			+6.0000			+159.1980
11 12			+8.0000			+212.2640
11 13			-12.0750			-255.2595
13 L			-12.0750			-255.2595
12 K			+9.8595			+343.9013
12 L *			+3.3000			+87.5589
K L			-5.4000			-35.8009

$\Delta JL = \frac{+125.0675A + 297.8787C + 480.2473}{E}$ inches.

Member.	U_3	$\frac{P_3U_3L}{A}$ A	$\frac{P_3U_3L}{A}$ C	$\frac{P_3U_3L}{A}$ #	$\frac{P_3L}{A}$
G J	-0.8533	+ 4.8273	+ 4.8273	+ 5.6572	+14.584
J I *	+1.7067	+ 32.0022	+ 77.2860	+ 90.5677	-17.250
G 1	+1.3146	+ 60.2790	+ 60.2790	-0-	-171.951
J 2	-1.9080	+ 76.9577	+ 76.9577	+ 90.7517	+103.689
2 3	-1.9080	+ 76.9577	+ 76.9577	+ 90.7517	+103.689
1 3	+1.0667	+ 30.1906	+ 30.1906	-0-	-106.135
1 4 *	+2.0667	+ 58.4934	+113.3291	+109.6715	- 53.069 + 42.451
3 4	-0.4770	+ 5.4077	+ 5.4077	-0-	-106.135
4 5	+0.5333	+15.0924	+ 15.0924	-0-	+ 28.063
3 6	-0.9540	+19.2394	+ 19.2394	+ 45.3759	+ 28.063
6 7	-0.9540	+19.2394	+ 19.2394	+ 45.3759	-106.135
5 7	+0.5333	+15.0924	+ 15.0924	-0-	-0-
4 8 *	+1.5333	+52.0710	+149.7103	+195.2786	+106.129
7 9	+0.5333	+15.0924	+ 15.0924	+113.2004	- 28.365
7 10	-0.9540	+19.2394	+ 19.2394	+ 99.2079	- 28.365
10 11	-0.9540	+19.2394	+ 19.2394	+ 99.2079	+106.129
8 9	+0.5333	+15.0924	+ 15.0924	+113.2004	-42.455
8 11	-0.4770	+ 5.4077	+ 5.4077	+ 40.5003	+ 53.063
8 12 *	+2.0667	+58.4934	+113.3291	+329.0145	+106.129
11 12	+1.0667	+30.1906	+ 30.1906	+228.4008	+104.006
11 13	-1.9080	+76.9577	+ 76.9577	+487.0351	-104.006
13 L	-1.9080	+76.9577	+ 76.9577	+487.0351	+171.950
12 K	+1.3146	+60.2790	+ 60.2790	+452.0926	+ 17.243
12 L *	+1.7067	+32.0022	+ 77.2860	+149.4368	- 14.586
K L	-0.8533	+ 4.8273	+ 4.8273	+ 30.5489	
		+879.6294	+1177.5077	+3300.3109	

$$\sum \frac{P_3U_3L}{A} = \frac{879.6294A + 1177.5077C + 3300.3109}{E}$$

Member.	U_5	$\frac{P_3U_5L}{A}$	$\frac{P_4U_1L}{A}$ A	$\frac{P_4U_1L}{A}$ C	$\frac{P_4U_1L}{A}$ R	$\frac{P_4L}{A}$
G J	+5.4000	+ 78.754	Same as A of $\frac{P_2U_1L}{A}$	Same as # of $\frac{P_2U_1L}{A}$	+ 4.7729	-0.0724
J I *	-3.3000	+ 56.925			+23.3505	+0.3758
G 1	-9.8595	+1695.351			-0-	+0.6516
J 2	+12.0750	+1252.045			+76.5778	-0.5161
2 3	+12.0750	+1252.045			+76.5778	-0.5161
1 3	- 8.0000	+ 849.080			-0-	+0.4021
1 4 *	- 6.0000	+ 318.414			+42.4554	+0.5114
3 4	+ 3.5777	+ 151.877			-0-	-0.1609
4 5	- 4.0000	+ 424.540			-0-	+0.4021
3 6	+ 4.9193	+ 138.050			+31.1972	-0.2295
6 7	+ 4.9193	+ 138.050			+31.1972	-0.2295
5 7	- 4.0000	+ 424.540			-0-	+0.4021
4 8 *	- 2.0000	-0-			+33.9665	+0.7450
7 9	-0-	-0-			-0-	+0.1474
7 10	+ 2.2500	- 63.821			+31.1972	-0.1618
10 11	+ 2.2500	- 63.821			+31.1972	-0.1618
8 9	-0-	-0-			-0-	+0.1474
8 11	-0-	-0-			-0-	-0.0590
8 12 *	- 2.0000	- 106.126			+42.4554	+0.3841
11 12	-0-	-0-			-0-	+0.1474
11 13	+ 2.2500	- 234.014			+76.5778	-0.2669
13 L	+ 2.2500	- 234.014			+76.5778	-0.2669
12 K	-0-	-0-			-0-	+0.2389
12 L *	[illegible].0000	- 34.486			+23.3505	+0.3344
K L	+ 1.0000	- 14.586			+ 4.7729	-0.0374
		+6028.803			+606.2241	

$$\sum \frac{P_3U_5L}{AE} = \frac{6,028.803}{E}$$

$$\sum \frac{P_4U_1L}{AE} = \frac{+376.0126A + 440.0494C + 606.2241R}{E}$$

Member		$\frac{1}{P}$	#	$\frac{P_2 l}{A}$
G	J	-3	+7.7331	-19.05
J	1	+4	+42.1131	+100.19
G	1	+4	+94.2375	+135.36
J	2	-7	+99.5428	-109.58
2	3	+7	+99.5428	-109.58
1	3	+4	+50.9434	+84.91
1	4	+5	+70.0471	+120.28
3	4	-1	+9.1142	-33.54
4	5	+2	+25.4717	+84.91
3	6	-3	+20.2773	-54.79
6	7	-3	+20.2733	-54.79
5	7	+2	+25.4717	+84.91
4	8	+3	+40.7547	+186.79
7	9	+2		+84.91
7	10	-3	+9.2167	-54.79
10	11	-3	+9.2167	-54.79
8	9	+2		+84.91
8	11	-1		-33.54
88	12	+5	+23.3480	+120.28
11	12	+4		+84.91
11	13	-7	+18.4334	-109.58
13	L	-7	+18.4334	-109.58
12	K	+4		+135.36
12	L	+4	+20.3773	+100.19
K	L	-3	+1.4321	-19.05
			+705.9853	

ΔL =

$\sum \frac{P_1 U}{A}$

es

TABLE XIV.

SPAN 40'- COL. HGT. 21'.

Member	P_1	$\frac{P_1 l}{A}$	U_1	$\frac{P_1 U_1 l}{A}$	P_2 A	P_2 #	$\frac{P_2 l}{A}$ A	$\frac{P_2 l}{A}$ #	$\frac{P_2 U_1 l}{A}$ A	$\frac{P_2 U_1 l}{A}$ #	$\frac{P_2 l}{A}$
G J	-3.370	-33.514	-0.8100	+27.415	-0.9600	-0.9600	-9.5470	-9.5470	+7.7331	+7.7331	-19.05
J 1	+4.230	+149.646	+0.6200	+93.529	+0.9200	+1.9200	+32.5471	+67.9244	+20.1792	+42.1131	+100.19
G 1	+4.860	+238.346	+1.3862	+232.493	+1.3862	+1.3862	+67.9826	+67.9826	+94.2375	+94.2375	+135.36
J 2	-7.495	-191.895	-1.8112	+249.248	-2.1466	-2.1466	-54.9596	-54.9596	+99.5428	+99.5428	-109.58
2 3	-7.495	-191.895	-1.8112	+249.248	-2.1466	-2.1466	-54.9596	-54.9596	+99.5428	+99.5428	-109.58
1 3	+4.220	+149.292	+1.2000	+180.793	+1.2000	+1.2000	+42.4528	+42.4528	+50.9434	+50.9434	+84.91
1 4	+5.195	+183.785	+0.9000	+164.[illegible]	+1.2000	+2.2000	+42.4528	+77.8301	+38.2075	+70.0471	+120.28
3 4	-1.875	- 59.328	-0.5367	+31.859	-0.5367	-0.5367	-16.9820	-16.9820	+9.1142	+9.1142	-33.54
4 5	+2.100	+148.585	+0.6000	+89.151	+0.6000	+0.6000	+42.4528	+42.4528	+25.4717	+25.4717	+84.91
3 6	-3.745	-95.884	-0.7379	+70.187	-1.0733	-1.0733	-27.4798	-27.4798	+20.2773	+20.2773	-54.79
6 7	-3.745	-95.884	-0.7379	+70.187	-1.0733	-1.0733	-27.4798	-27.4798	+20.2773	+20.2733	-54.79
5 7	+2.100	+148.585	+0.6000	+89.151	+0.6000	+0.6000	+42.4528	+42.4528	+25.4717	+25.4717	+84.91
4 8	+3.100	+263.208	+0.3000	+76.857	+0.6000	+1.6000	+50.9434	+135.8491	+15.2830	+40.7547	+186.79
7 9	+2.100	+148.585			+0.6000	+0.6000	+42.4528	+42.4528			+84.91
7 10	-3.745	-95.884	-0.3354	+31.642	-1.0733	-1.0733	-27.4798	-27.4798	+9.2167	+9.2167	-54.79
10 11	-3.475	-95.884	-0.3354	+31.642	-1.0733	-1.0733	-27.4798	-27.4798	+9.2167	+9.2167	-54.79
8 9	+2.100	+148.585			+0.6000	+0.6000	+42.4528	+42.4528			+84.91
8 11	-1.875	-59.328			-0.5367	-0.5367	-16.9820	-16.9820			-33.54
88 12	+5.195	+183.785	+0.3000	+53.665	+1.2000	+2.2000	+42.4528	+77.8301	+12.7358	+23.3480	+120.28
11 12	+4.220	+149.292			+1.2000	+1.2000	+42.4528	+42.4528			+84.91
11 13	-7.495	-191.895	-0.3354	+63.325	-2.1466	-2.1466	-54.9596	+54.9596	+18.4334	+18.4334	-109.58
13 L	-7.495	-191.895	-0.3354	+63.325	-2.1466	-2.1466	-54.9596	-54.9596	+18.4334	+18.4334	-109.58
12 K	+4.860	+238.346			+1.3862	+1.3862	+67.9826	+67.9826			+135.36
12 L	+4.230	+149.646	+0.3000	+43.697	+0.9200	+1.9200	+32.5471	+67.9244	+9.7641	+20.3773	+100.19
K L	-3.370	-33.514	-0.1[illegible]00	+ 5.027	-0.9600	-0.9600	- 9.5470	-9.5470	+1.4321	+1.4321	-19.05
				+2216.927					+605.5137	+705.9853	

$$\Delta JL = \frac{930.070}{E} \text{ inches}$$

$$\sum \frac{P_1 U_1 l}{A\,E} = \frac{2216.927}{E} \text{ inches.}$$

$$\Delta JL = \frac{200.9432A + 427.3581}{E} \text{ inches}$$

$$\sum \frac{P_2 U_1 l}{AE} = \frac{605.5137A + 705.9853}{E} \text{ inches.}$$

$\frac{P_3\,1}{A}$	U_5	$\frac{P_3\,U_5\,1}{A}$
+21.878	+5.4000	+118.141
-37.735	-4.1333	+155.970
226.609	-9.2413	+2094.162
125.948	+12.0748	+1520.797
125.948	+12.0748	+1520.797
141.509	-8.0000	+1132.072
-70.755	-6.0000	+424.530
+56.607	+3.5777	
141.509	-4.0000	+566.036
+34.348	+4.9194	+168.972
+34.348	+4.9194	+168.972
141.509	-4.0000	+566.036
0	-2.0000	
141.509		
-34.348	+2.2361	-76.806
-34.348	+2.2361	-76.806
141.509		
-56.607		
+70.755	-2.0000	-141.510
141.509		
12[illegible].948	+2.2361	-281.632
1[illegible].948	+2.2361	-281.632
226.609		
+37.735	-2.0000	-75.470
-21.878	+1.0000	-21.878
		+7480 .751

$\frac{[illegible]\,U\,1}{[illegible]\,E} = \frac{74[illegible].7510}{E}$ inches

TABLE XV.

SPAN 40' - COL. HGT. 21'.

Member	P_3 A	P_3 C	P_3 #	$\frac{P_3 L}{A}$ A	$\frac{P_3 L}{A}$ C	$\frac{P_3 L}{A}$ #	U_3	$\frac{P_3 U_3 l}{A}$ A	$\frac{P_3 U_3 l}{A}$ C	$\frac{P_3 U_3 l}{A}$
6 J			-1.0000			-9.9448	-0.9600	+9.[illegible]651	+9.1[illegible]51	+[illegible].54[illegible]0
J 1			+2.0000			+70.7546	+1.92[illegible]0	+62.4[illegible]04	+13[illegible].4148	+13[illegible].8488
5 1							+1.3862	+94.[illegible]75	+94.2[illegible]7[illegible]	
J 2			+2.2361			-57.2511	-[illegible].146[illegible]	+117.9763	+1[illegible]7.[illegible]763	+1[illegible].[illegible]052
2 3	Same as	Same as	-2.2361	Same as	Same as	-57.2511	-[illegible].1466	+117.[illegible]763	+117.[illegible]7[illegible]	+12[illegible].[illegible]52
1 3							+1.2000	+50.9434	+50.9434	
1 4	A of P_2	# of P_2	+2.0000	A of $P_2 l/A$	# of $P_2 l/A$	+70.7546	+2.2000	+93.3962	+171.2262	+155.6601
3 4							-0.5367	+9.1142	+9.1142	
4 5							+0.6000	+25.4717	+25.4717	
3 6			-2.2361			-57.2511	-1.0733	+29.4941	+29.4941	+61.4476
6 7			-2.2361			-57.2511	-1.0733	+29.4941	+29.4941	+61.4476
5 7							+0.6000	+25.4717	+25.4717	
4 8			+2.0000			+169.8114	+1.6000	+81.5094	+217.3586	+271.6982
7 9			+4.0000			+283.0188	+0.6000	+25.4717	+25.4717	+169.8113
7 10			-4.9194			+125.9519	-1.0733	+29.4941	+29.4941	+135.1842
10 11			-4.9194			-125.9519	-1.0733	+29.4941	+29.4941	+135.1842
8 9			+4.0000			+283.0188	+0.6000	+25.4717	+25.4717	+169.8113
8 11			-3.5777			-113.2038	-0.5367	+9.1142	+9.1142	+60.7565
8 12			+6.0000			+212.2638	+2.2000	+93.3962	+171.2262	+466.9[illegible]04
11 12			+8.0000			+283.0184	+1.2000	+50.9434	+50.9434	+339.6221
11 13			-12.0748			-309.1523	-2.1466	+117.9763	+117.9763	+663.6263
13 L			-12.0748			-309.1523	-2.1466	+117.9763	+117.9763	+663.6263
12 K			+9.2413			+453.2155	+1.3862	+94.2375	+94.2375	+628.2473
12 L			+4.1333			+146.2250	+1.9200	+62.4904	+130.4148	+280.7520
I L			-5.4000			-53.7019	-0.9600	+9.1651	+9.1651	+51.5538
								+1411.9714	+1839.3294	+4706.5954

Member	$\frac{P_3 l}{A}$	U_5	$\frac{P_3 U_5 l}{A}$	$\frac{P_4 U_1 l}{A}$ A	$\frac{P_4 U_1 l}{A}$ C	$\frac{P_4 U_1 l}{A}$ R	$\frac{P_4 l}{A}$
6 J	+21.878	+5.4000	+118.141			8.0553	-.[illegible]910
J 1	-37.735	-4.1333	+155.970			43.8679	+.4606
5 1	-226.609	-9.2413	+2094.162				+.7131
J 2	+125.948	+12.0748	+1520.797			103.6920	-.5238
2 3	+125.948	+12.0748	+1520.797	Same as	Same as	103.6920	-.5238
1 3	-141.509	-8.0000	+1132.072				+.4454
1 4	-70.755	-6.0000	+424.530	A of $\frac{P_2 U_1 l}{A}$	# of $\frac{P_2 U_1 l}{A}$	63.6791	+.5646
3 4	+56.607	+3.5777					-.1782
4 5	-141.509	-4.0000	+566.036				+.4454
3 6	+34.348	+4.9194	+168.972			42.2450	-.2358
6 7	+34.348	+4.9194	+168.972			42.2450	-.2358
5 7	-141.509	-4.0000	+566.036				+.44[illegible]4
4 8	0	-2.0000				50.9434	+.8206
7 9	+141.509						+.1[illegible]50
7 10	-34.348	+2.2361	-76.806			42.2450	-.1724
10 11	-34.348	+2.2361	-76.806			42.2450	-.1724
8 9	+141.509						+.18[illegible]0
8 11	-56.607						-.[illegible]741
8 12	+70.755	-2.0000	-141.510			63.6791	+.4344
11 12	+141.509						+.1850
11 13	-12[illegible].948	+2.2361	-281.632			103.6920	-.2921
13 L	-1[illegible].948	+2.2361	-281.632			103.6920	-.[illegible]921
12 K	+226.609						+.2961
12 L	+37.735	-2.0000	-75.470			43.8679	+.3912
I L	-21.878	+1.0000	-21.878			8.0553	-.0507
			+7480.751	+605.5137	705.9853	865.8960	

$\Delta JL = \frac{200.9432A + 427.3581C + 670.0936}{E}$ inches

$\sum \frac{P_3 U_3 l}{AE} = \frac{1411.[illegible]714A + 1[illegible]9.[illegible]29[illegible]C + 47[illegible].[illegible]}{E}$ in.

$\sum \frac{P_3 U_5 l}{AE} = \frac{74[illegible].7[illegible]10}{E}$ inches

$\sum \frac{P_4 U_1 l}{AE} = \frac{605.5137A + 7[illegible]5.98[illegible]3C + [illegible]5.8360R}{E}$ in.

L 1 A #	P L 2 A
+ 4.9492	- 15.527
+ 17.9093	+ 82.314
+78.9641	+127.317
+58.6113	- 82.234
+58.6113	- 82.234
+40.7547	+ 86.306
+55.5883	+116.269
+ 7.2896	- 34.520
+20.3773	+ 86.306
+11.9386	- 41.116
+11.9386	- 41.116
+20.3773	+ 86.306
+37.6981	+209.700
-0-	+ 86.306
+ 5.4263	- 41.116
+ 5.4263	- 41.116
-0-	+ 86.306
-0-	- 34.520
+18.5294	+116.269
-0-	+ 86.306
+10.8526	- 82.234
+10.8526	- 82.234
-0-	+127.317
+14.5210	+ 82.314
+ 0.9165	- 15.527
+491.5324	

$\frac{407.1321A + 491.5324}{E}$ inches.

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TABLE XVI.

SPAN 50' - COL. HGT. 21'.

Member.	P_1	$\frac{P_1 L}{A}$	U_1	$\frac{P_1 U_1 L}{A}$	P_2		$\frac{P_2 L}{A}$		$\frac{P_2 U_1 L}{A}$		$\frac{P_2 L}{A}$
					A	#	A	#	A	#	
G J	-2.6880	- 26.7316	-0.6480	+17.3221	-0.7680	-0.7680	- 7.6376	- 7.6376	+ 4.9492	+ 4.9492	- 15.527
J 1	+2.8760	+113.2879*	+0.2960	+33.5332	+0.5360	+1.5360	+21.1135	+60.5043	+ 6.2496	+17.9093	+ 82.314
G 1	+4.4143	+219.2455	+1.2609	+276.4467	+1.2609	+1.2609	+62.6252	+62.6252	+78.9641	+78.9641	+127.317
J 2	-6.0105	-141.5719	-1.4490	+205.1377	-1.7173	-1.7173	-40.4495	-40.4495	+58.6113	+58.6113	- 82.234
2 3	-6.0105	-141.5719	-1.4490	+205.1377	-1.7173	-1.7173	-40.4495	-40.4495	+58.6113	+58.6113	- 82.234
1 3	+3.3600	+148.5849	+0.9600	+142.6415	+0.9600	+0.9600	+42.4528	+42.4528	+40.7547	+40.7547	+ 86.306
1 4	+4.3600	+171.7439*	+0.7200	+123.6556	+0.9600	+1.9600	+37.8152	+77.2060	+27.2269	+55.5883	+116.269
3 4	-1.5026	- 59.4325	-0.4293	+ 25.5144	-0.4293	-0.4293	-16.9801	-16.9801	+ 7.2896	+ 7.2896	-. 34.520
4 5	+1.6800	+148.5849	+0.4800	+ 71.3208	+0.4800	+0.4800	+42.4528	+42.4528	+20.3773	+20.3773	+ 86.306
3 6	-3.0053	- 70.7860	-0.5903	+ 41.7850	-0.8587	-0.8587	-20.2247	-20.2247	+11.9386	+11.9386	- 41.116
6 7	-3.0053	- 70.7860	-0.5903	+ 41.7850	-0.8587	-0.8587	-20.2247	-20.2247	+11.9386	+11.9386	- 41.116
5 7	+1.6800	+148.5849	+0.4800	+ 71.3208	+0.4800	+0.4800	+42.4528	+42.4528	+20.3773	+20.3773	+ 86.306
4 8	+2.6800	+284.4340*	+0.2400	+ 68.2642	+0.4800	+1.4800	+50.9434	+157.0755	+12.2264	+37.6981	+209.700
7 9	- -	+148.5849	-0-	-0-					-0-	-0-	+ 86.306
7 10	- -	- 70.7860	-0.2683	+ 18.9919					+ 5.4263	+ 5.4263	- 41.116
10 11	- -	- 70.7860	-0.2683	+ 18.9919					+ 5.4263	+ 5.4263	- 41.116
8 9	- -	+148.5849	-0-	-0-					-0-	-0-	+ 86.306
8 11	Symmetrical	- 59.4325	-0-	-0-					-0-	-0-	- 34.520
8 12	- -	+171.7439*	+0.2400	+ 41.2185	Symmetrical.		Symmetrical.		+ 9.0756	+18.5294	+116.269
11 12	- -	+148.5849	-0-	-0-					-0-	-0-	+ 86.306
11 13	- -	-141.5719	-0.2683	+ 37.9837					+10.8526	+10.8526	- 82.234
13 L	- -	-141.5719	-0.2683	+ 37.9837					+10.8526	+10.8526	- 82.234
12 K	- -	+219.2455	-0-	-0-					-0-	-0-	+127.317
12 L	- -	+113.2879*	+0.2400	+ 27.1891					+ 5.0673	+14.5210	+ 82.314
K L	- -	- 26.7316	-0.1200	+ 3.2078					+ 0.9165	+ 0.9165	- 15.527
				+1509.4313					+407.1321	+491.5324	

$$\Delta JL = \frac{854.4976}{E} \text{ inches.}$$

$$\sum \frac{P_1 U_1 L}{AE} = \frac{1509.4313}{E} \text{ inches.}$$

$$\Delta JL = \frac{168.8008\,A + 432.4961}{E} \text{ inches.}$$

$$\sum \frac{P_2 U_1 L}{AE} = \frac{407.1321A + 491.5324}{E} \text{ inches.}$$

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U_5	$\frac{P U_3 L_5}{A}$	$\frac{P U_4 L_1}{A}$ A	$\frac{P U_4 L_1}{A}$ C	$\frac{P U_4 L_1}{A}$ R	$\frac{P L_4}{A}$
+5.4000	+ 118.1223			+ 6.4442	-0.0788
-2.4667	+ 22.6722			+23.3195	+0.3992
-10.5075	+2741.8102			-0-	+0.7036
+12.0750	+1399.2184	Same.	Same.	+76.3082	-0.4176
+12.0750	+1399.2184			+76.3082	-0.4176
- 8.0000	+1415.0936	As A	As #	-0-	+0.4769
- 6.0000	+ 472.6902	of	of	+56.7228	+0.5868
+ 3.5775	+ 253.1096			-0-	-0.1908
- 4.0000	+ 707.5468	$\frac{P_2U_1L}{A}$	$\frac{P_2U_1L}{A}$	-0-	+0.4769
+ 4.9192	+ 155.4826			+31.0869	-0.1903
+ 4.9192	+ 155.4826			+31.0869	-0.1903
- 4.0000	+ 707.5468			-0-	+0.4769
- 2.0000	-0-			+50.9434	+1.0086
-0-	-0-			-0-	+0.2293
+ 2.2358	- 70.6461			+31.0869	-0.1461
+ 2.2358	- 70.6461			+31.0869	-0.1461
-0-	-0-			-0-	+0.2293
-0-	-0-			-0-	-0.0917
2.0000	-157.5630			+56.7228	+0.4765
-0-	-0-			-0-	+0.2293
+ 2.2358	-259.0760			+76.3082	-0.2554
- 2.2358	-259.0760			+76.3082	-0.2554
-0-	-0-			-0-	+0.3383
2.0000	- 18.3828			+23.3195	+0.3863
- 1.0000	- 21.8786			+ 6.4442	-0.0482
	+8690.7251			+653.4968	

$$\sum \frac{P U_3 L_5}{AE} = \frac{8,690.7251}{E}$$

$$\sum \frac{P_4 U_1 L}{AE} = \frac{407.1321 A + 491.5324 C + 653.4968 R}{E}$$

TABLE XVII.

SPAN 50'-COL. HGT. 21'.

Member.	P_3 A	P_3 C	P_3 #	$\frac{PL_3}{A}$ A	$\frac{PL_3}{A}$ C	$\frac{PL_3}{A}$ #	U_3	$\frac{PU_3L_3}{A}$ A	$\frac{PU_3L_3}{A}$ C	$\frac{PU_3L_3}{A}$ #	$\frac{PL_3}{A}$	U_5	$\frac{PU_3L_5}{A}$	$\frac{PU_4L_1}{A}$ A	$\frac{PU_4L_1}{A}$ C	$\frac{PU_4L_1}{A}$ R	$\frac{PL_4}{A}$
G J			-1.0000			-9.9448	-0.7680	+ 5.8657	+ 5.8657	+ 7.6376	+ 21.8745	+5.4000	+ 118.1223			+ 6.4442	-0.0788
J 1			+2.0000			+78.7816	+1.5360	+32.4303	+92.9346	+121.0085	- 9.1913	-2.4667	+ 22.6722			+23.3195	+0.3992
G 1			-0-			-0-	+1.2609	+78.8380	+78.8380	-0-	-260.9384	-10.5075	+2741.8102			-0-	+0.7036
J 2	Same. as A of P	Same. as # of P	-2.2358	Same. as A of $\frac{PL_2}{A}$	Same. as # of $\frac{PL_2}{A}$	-52.6623	-1.7173	+69.4639	+69.4639	+ 90.4370	+115.8773	+12.0750	+1399.2184	Same. As A of $\frac{P_2U_1L}{A}$	Same. As # of $\frac{P_2U_1L}{A}$	+76.3082	-0.4176
2 3			-2.2358			-52.6623	-1.7173	+69.4639	+69.4639	+ 90.4370	+115.8773	+12.0750	+1399.2184			+76.3082	-0.4176
1 3			-0-			-0-	+0.9600	+40.7547	+40.7547	-0-	-176.8867	- 8.0000	+1415.0936			-0-	+0.4769
1 4			+2.0000			+78.7816	+1.9600	+74.1178	+151.3238	+154.4119	- 78.7817	- 6.0000	+ 472.6902			+56.7228	+0.5868
3 4			-0-			-0-	-0.4293	+ 7.2896	+ 7.2896	-0-	+ 70.7504	+ 3.5775	+ 253.1096			-0-	-0.1908
4 5			-0-			-0-	+0.4800	+20.3773	+20.3773	-0-	-176.8867	- 4.0000	+ 707.5468			-0-	+0.4769
3 6			-2.2358			-52.6623	-0.8587	+17.3659	+17.3659	+45.2185	+ 31.6073	+ 4.9192	+ 155.4826			+31.0869	-0.1903
6 7			-2.2358			-52.6623	-0.8587	+17.3659	+17.3659	+45.2185	+ 31.6073	+ 4.9192	+ 155.4826			+31.0869	-0.1903
5 7			-0-			-0-	+0.4800	+20.3773	+20.3773	-0-	-176.8867	- 4.0000	+ 707.5468			-0-	+0.4769
4 8			+2.0000			+212.2642	+1.4800	+75.3962	+232.4717	+314.1510	-0-	- 2.0000	-0-			+50.9434	+1.0086
7 9			+4.0000			+353.7736	+0.4800	+20.3773	+ 20.3773	+169.8113	+176.8869	-0-	-0-			-0-	+0.2293
7 10			-4.9192			-115.8673	-0.8587	+17.3659	+ 17.3659	+ 99.4895	- 31.5977	+ 2.2358	- 70.6461			+31.0869	-0.1461
10 11			-4.9192			-115.8673	-0.8587	+17.3659	+ 17.3659	+ 99.4895	- 31.5977	+ 2.2358	- 70.6461			+31.0869	-0.1461
8 9			+4.0000			+353.7736	+0.4800	+20.3773	+ 20.3773	+169.8113	+176.8869	-0-	-0-			-0-	+0.2293
8 11			-3.5775			-141.5012	-0.4293	+ 7.2896	+ 7.2896	+ 60.7465	- 70.7508	-0-	-0-			-0-	-0.0917
8 12			+6.0000			+236.3448	+2.9600	+74.1178	+151.3238	+463.2358	+ 78.7815	- 2.0000	-157.5630			+56.7228	+0.4765
11 12			+8.0000			+353.7736	+0.9600	+40.7547	+ 40.7547	+339.6227	-176.8869	-0-	-0-			-0-	+0.2293
11 13			-12.0750			-284.4158	-1.7173	+69.4639	+ 69.4639	+488.4273	-115.8762	+ 2.2358	-259.0760			+76.3082	-0.2554
13 L			-12.0750			-284.4158	-1.7173	+69.4639	+ 69.4639	+488.4273	-115.8762	+ 2.2358	-259.0760			+76.3082	-0.2554
12 K			+10.5075			+521.8771	+1.2609	+78.8380	+ 78.8380	+658.0348	+260.9387	-0-	-0-			-0-	+0.3383
12 L			+ 2.4667			+ 97.1653	+1.5360	+32.4303	+ 92.9346	+149.2459	+ 9.1914	- 2.0000	- 18.3828			+23.3195	+0.3863
K L			- 5.4000			- 53.7019	-0.7680	+ 5.8657	+ 5.8657	+ 41.2430	- 21.8786	+ 1.0000	- 21.8786			+ 6.4442	-0.0482
								+982.8168	+1415.3129	+4096.1049			+8690.7251			+653.4968	

$$\Delta J\,L = \frac{168.8008\,A + 432.4961\,C + 703.3375}{E}$$

$$\sum \frac{PU_3L_3}{AE} = \frac{982.8168A + 1415.3129C + 4096.1049}{E}$$

$$\sum \frac{PU_3L_5}{AE} = \frac{8,690.7251}{E}$$

$$\sum \frac{P_2U_1L}{AE} = \frac{407.1321\,A + 491.5324\,C + 653.4968\,R}{E}$$

#	$\frac{P_2\ 1}{A}$
+3.4369	-13.04
+4.3969	+67.54
+71.8654	+123.15
+45.0471	-76.31
+45.0471	-76.31
+33.9622	+86.90
+46.3740	+113.25
+6.0764	-34.98
+15.1260	+77.42
+9.1761	-38.42
+9.1761	-38.42
+15.1260	+77.42
+35.6604	+231.63
	+77.42
+4.1711	-38.42
+4.1711	-38.42
	+77.42
	-34.98
+15.4580	+113.25
	+86.90
+8.3416	-76.31
+8.3416	-76.31
	+123.15
+10.9924	+67.54
+0.6365	-13.04
+392.5829	

TABLE XVIII

SPAN [illegible] COL HGT 2[illegible]

Member	P_1	$\frac{P_1 l}{A}$	U_1	$\frac{P_1 U_1 l}{A}$	P_2		$\frac{P_2 l}{A}$		$\frac{P_2 U_1 l}{A}$		$\frac{P_2 l}{A}$
					A	#	A	#	A	#	
G J	-2.25	-22.38	-0.5400	+12.11	-0.6400	-0.6400	-6.3647	-6.3647	+3.4369	+3.4369	-13.04
J 1	+2.00	+85.88	+0.0800	+7.30	+0.2800	+1.2800	+12.0229	+54.9618	+0.9618	+4.3969	+67.54
G 1	+4.15	+211.57	+1.1873	+251.77	+1.1873	+1.1873	+60.5284	+60.5284	+71.8654	+71.8654	+123.15
J 2	-5.03	-131.12	-1.2075	+158.92	-1.4311	-1.4311	-37.3061	-37.3061	+45.0471	+45.0471	-76.31
2 3	-5.03	-131.12	-1.2075	+158.92	-1.4311	-1.4311	-37.3061	-37.3061	+45.0471	+45.0471	-76.31
1 3	+2.80	+148.58	+0.8000	+120.06	+0.8000	+0.8000	+42.4528	+42.4528	+33.9622	+33.9622	+86.90
1 4	+3.80	+163.17	+0.6000	+ 97.08	+0.8000	+1.8000	+34.3511	+77.2900	+20.6107	+46.3740	+113.25
3 4	-1.25	- 59.33	-0.3578	+21.12	-0.3578	-0.3578	-16.9826	-16.9826	+6.0764	+6.0764	-34.98
4 5	+1.40	+132.35	+0.4000	+52.28	+0.4000	+0.4000	+37.8151	+37.8151	+15.1260	+15.1260	+77.42
3 6	-2.50	-65.17	-0.4919	+31.74	-0.7156	-0.7156	-18.6543	-18.6543	+9.1761	+9.1761	-38.42
6 7	-2.50	-65.17	-0.4919	+31.74	-0.7156	-0.7156	-18.6543	-18.6543	+9.1761	+9.1761	-38.42
5 7	+1.40	+132.35	+0.4000	+52.28	+0.4000	+0.4000	+37.8151	+37.8151	+15.1260	+15.1260	+77.42
4 8	+2.40	+305.66	+0.2000	+59.60	+0.4000	+1.4000	+50.9434	+178.3019	+10.1887	+35.6604	+231.63
7 9	+1.40	+132.35			+0.4000	+0.4000	+37.8151	+37.8151			+77.42
7 10	-2.50	-65.17	-0.2236	+14.34	-0.7156	-0.7156	-18.6543	-18.6543	+4.1711	+4.1711	-38.42
10 11	-2.50	-65.17	-0.2236	+14.34	-0.7156	-0.7156	-18.6543	-18.6543	+4.1711	+4.1711	-38.42
8 9	+1.40	+132.35			+0.4000	+0.4000	+37.8151	+37.8151			+77/42
8 11	-1.25	-59.33			-0.3578	-0.3578	-16.9826	-16.9826			-34.98
8 12	+3.80	+163.17	+0.2000	+31.82	+0.8000	+1.8000	+34.3511	+77.2900	+6.8702	+15.4580	+113.25
11 12	+2.80	+148.58			+0.8000	+0.8000	+42.4528	+42.4528			+86.90
11 13	-5.03	-131.12	-0.2236	+28.85	-1.4311	-1.4311	-37.3061	-37.3061	+8.3416	+8.3416	-76.31
13 L	-5.03	-131.12	-0.2236	+28.85	-1.4311	-1.4311	-37.3061	-37.3061	+8.3416	+8.3416	-76.31
12 K	+4.15	+211.57			+1.1873	+1.1873	+60.5284	+60.5284			+123.15
12 L	+2.00	+85.88	+0.2000	+16.75	+0.2800	+1.2800	+12.0229	+54.9618	+2.4046	+10.9924	+67.54
K L	-2.25	-22.38	-0.1000	+2.24	-0.6400	-0.6400	-6.3647	-6.3647	+0.6365	+0.6365	-13.04
				+1191.98					+320.7372	+392.5829	

$\Delta_{JL} = \frac{80[illegible]}{E}$ inches. $\sum \frac{P_1 U_1 l}{AE} = \frac{11[illegible]8}{E}$ inches

$\Delta_{JL} = \frac{143.6914A + 442.[illegible]066}{E}$ inches

$\sum \frac{P_2 U_1 l}{A E} = \frac{320.7372A + 392.5829}{E}$ inches

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TABLE XIX.

SPAN 60'- COL. HGT. 21

Member	P_3 A	P_3 C	P_3 #	$\frac{P_3 l}{A}$ A	$\frac{P_3 l}{A}$ C	$\frac{P_3 l}{A}$ #
G J			-1.0			-9.9448
J 1			+2.0			+85.8778
G 1						
J 2	Same as A of P_2	Same as # of P_2	-2.2361	Same as A of $\frac{P_2 l}{A}$	Same as # of $\frac{P_2 l}{A}$	-58.2909
2 3			-2.2361			-58.2909
1 3						
1 4			+2.0			+85.8778
3 4						
4 5						
3 6			-2.2361			-58.2909
6 7			-2.2361			-58.2909
5 7						
4 8			+2.0			+254.7170
7 9			+4.0			+378.1512
7 10			-4.9194			-128.2394
10 11			-4.9194			-128.2394
8 9			+4.0			+378.1512
8 11			-3.5377			+169.8018
8 12			+6.0			+257.6334
11 12			+8.0			+424.5280
11 13			-12.0748			-314.7671
13 L			-12.0748			-314.7671
12 K			+11.8727			+605.2691
12 L			+0.80			+34.3511
K L			-5.40			-53.7019

ΔˈL [illegible].6914A + 1[illegible]C + 71[illegible] / E inches

Member	U_3	$\frac{P_3 U_3 l}{A}$ A	$\frac{P_3 U_3 l}{A}$ C	$\frac{P_3 U_3 l}{A}$ #
G J	-0.6400	4.0734	4.0734	6.3647
J 1	+1.2800	15.3893	70.3511	109.9236
G 1	+1.1873	71.8654	71.8654	
J 2	-1.4311	53.3888	53.3888	83.4201
2 3	-1.4311	53.3888	53.3888	83.4201
1 3	+0.8000	33.9622	33.9622	
1 4	+1.8000	61.8320	139.1220	154.5800
3 4	-0.3578	6.0764	6.0764	
4 5	+0.4000	15.1260	15.1260	
3 6	-0.7156	13.3481	13.3481	41.7130
6 7	-0.7156	13.3481	13.3481	41.7130
5 7	+0.4000	15.1260	15.1260	
4 8	+1.4000	71.3208	249.6227	356.6038
7 9	+0.8000	15.1260	15.1260	151.2605
7 10	-0.7156	13.3481	13.3481	91.7681
10 11	-0.7156	13.3481	13.3481	91.7681
8 9	+0.4000	15.1260	15.1260	151.2605
8 11	-0.3578	6.0764	6.0764	60.7550
8 12	+1.8000	61.8320	139.1220	463.7401
11 12	+0.8000	33.9622	33.9622	339.6224
11 13	-1.4311	53.3888	53.3888	450.4632
13 L	-1.4311	53.3888	53.3888	450.4632
12 K	+1.1873	71.8654	71.8654	718.6360
12 L	+1.2800	15.3893	70.3511	43.9694
K L	-0.6400	4.0734	4.0734	34.3692
		785.1698	1227.9699	3925.8140

$\sum \frac{P_3 U_3 l}{AE} = \frac{785.1698A + 1227.9699C + 3925.8140}{E}$ inches

Member	$\frac{P_3 l}{A}$	U_5	$\frac{P_3 U_5 l}{A}$
G J	+21.8785	+5.4	+118.1439
J 1	+25.7634	-0.8	-20.6107
G 1	-302.6420	-11.8727	+3593.1777
J 2	+128.2380	+12.0748	+1548.4482
2 3	+128.2380	+12.0748	+1548.4482
1 3	-212.2640	-8.0	+1698.1120
1 4	-85.8778	-6.0	+515.2668
3 4	+ 84.9130	+3.5777	+303.7932
4 5	-189.0756	-4.0	+756.3024
3 6	+34.9730	+4.9194	+172.0462
6 7	+34.9730	+4.9194	+172.0462
5 7	-189.0756	-4.0	+756.3024
4 8	0	-2.0	
7 9	+189.0756		
7 10	-34.9730	+2.2361	- 78.2031
10 11	-34.9730	+2.2361	-78.2031
8 9	+189.0756		
8 11	-84.9130		
8 12	+85.8778	-2.0	-171.7556
11 12	+212.2640		
11 13	-128.2380	+2.2361	-286.7530
13 L	-128.2380	+2.2361	-286.7530
12 K	+302.6420		
12 L	-25.7634	-2.0	+51.5268
K L	-21.8785	+1.0	-21.8785
			+10,289.4570

$\sum \frac{P_3 U_5 l}{AE} = \frac{1[illegible],2[illegible].4570}{E}$ inches

Member	$\frac{P_4 U_1 l}{A}$ A	$\frac{P_4 U_1 l}{A}$ C	$\frac{P_4 U_1 l}{A}$ R	$\frac{P_4 l}{A}$
G J			5.3702	-0.0677
J 1			6.8702	+0.3310
G 1				+0.6985
J 2	Same as A of $\frac{P_2 U_1 l}{A}$	Same As # of $\frac{P_2 U_1 l}{A}$	70.3840	-0.3971
2 3			70.3840	-0.3971
1 3				+0.4899
1 4			51.5267	+0.5887
3 4				-0.1960
4 5				+0.4364
3 6			28.6738	-0.1818
6 7			28.6738	-0.1818
5 7				+0.4364
4 8			50.9434	+1.1581
7 9				+0.2193
7 10			28.6738	-0.1419
10 11			28.6738	-0.1419
8 9				+0.2193
8 11				-0.0985
8 12			51.5267	+0.4901
11 12				+0.2462
11 13			70.3840	-0.2498
13 L			70.3840	-0.2498
12 K				+0.3511
12 L			6.8702	+0.3606
K L			5.3702	-0.0426
	+320.7372	+392.5829	+574.7088	

$\sum \frac{P_4 U_1 l}{AE} = \frac{320.[illegible]72A + 39[illegible] [illegible] 74.7[illegible]8R}{E}$ in.

TABLE XX.

Member	SPAN 20 FT.			Column height 10 ft.			Column height 16 ft.			Column height 19 ft.		
	$\frac{PL}{A}$			$\frac{P_1L}{A}$	$\frac{P_2L}{A}$	$\frac{P_4L}{A}$	$\frac{P_1L}{A}$	$\frac{P_2L}{A}$	$\frac{P_4L}{A}$	$\frac{P_1L}{A}$	$\frac{P_2L}{A}$	$\frac{P_4L}{A}$
	A	C	R	A = +1.500 C = +1.000 R = 0	A = +0.388 C = +1.000 R = 0	A = +0.00403 C = +0.00961 R = -0.00209	A = +3.000 C = +1.000 R = 0	A = +1.162 C = +1.000 R = 0	A = +0.00766 C = +0.00646 R = -0.00182	A = +3.750 C = +1.000 R = 0	A = +1.543 C = +1.000 R = 0	A = +0.00856 C = +0.00547 R = -0.00178
G J	- 3.72	- 3.72	- 5.81	- 9.30	- 6.16	-0.039	- 14.88	- 8.04	-0.041	- 17.66	- 9.46	-0.042
G T	+ 49.87	+ 49.87	0	+ 124.67	+ 69.22	+0.679	+199.47	+107.77	+0.704	+ 235.87	+ 126.77	+0.700
J T	+ 8.82	+ 40.34	+ 63.03	+ 53.57	+ 43.76	+0.292	+ 66.81	+ 50.59	+0.214	+ 73.39	+ 53.94	+0.184
J S	- 73.28	- 73.28	- 114.50	- 183.20	- 101.71	-0.760	-293.13	-158.38	-0.826	- 348.28	- 186.88	-0.824
T S	+ 56.60	+ 56.60	0	+ 141.51	+ 78.58	+0.772	+226.42	+122.30	+0.800	+ 268.60	+ 143.90	+0.795
T U	+ 30.25	+ 68.07	+ 75.63	+ 113.45	+ 79.80	+0.622	+158.82	+103.25	+0.534	+ 181.37	+ 113.77	+0.496
S U			+ 283.02			+0.180			+0.285			+0.291
S L			- 251.91			-0.473			-0.576			-0.580
U L	Symmetrical		- 18.91	Symmetrical		+0.464	Symmetrical		+0.363	Symmetrical		+0.262
U K			+ 249.34			+0.158			+0.250			+0.256
L K			- 12.78			-0.024			-0.029			-0.029

Membe	
G	J
J	1
G	1
J	2
2	3
1	3
1	4
3	4
4	5
3	6
6	7
5	7
4	8
7	9
7	10
10	11
8	9
8	11
8	12
11	12
11	13
13	L
12	K
12	L
K	L

TABLE XXI.

Member	SPAN 30 FT. $\frac{PL}{\Lambda}$			Column height 21 ft. $\frac{P_1L}{\Lambda}$	Column height 21 ft. $\frac{P_2L}{\Lambda}$	Column height 21 ft. $\frac{P_4L}{\Lambda}$	Column height 26 ft. $\frac{P_1L}{\Lambda}$	Column height 26 ft. $\frac{P_2L}{\Lambda}$	Column height 26 ft. $\frac{P_4L}{\Lambda}$	Column height 31 ft. $\frac{P_1L}{\Lambda}$	Column height 31 ft. $\frac{P_2L}{\Lambda}$	Column height 31 ft. $\frac{P_4L}{\Lambda}$
	A	C	R	A = +4.250 C = +1.000 R = 0	A = +1.792 C = +1.000 R = 0	A = +0.00907 C = +0.00500 R = -0.00115	A = +5.500 C = +1.000 R = 0	A = +2.423 C = +1.000 R = 0	A = +0.00974 C = +0.00398 R = -0.00115	A = +6.750 C = +1.000 R = 0	A = +2.795 C = +1.000 R = 0	A = +0.00880 C = +0.00308 R = -0.00118
0 J	- 5.66	- 5.66	- 6.63	- 29.7	- 17.56	-0.071	- 36.8	- 19.38	-0.070	- 43.9	- 17.78	-0.047
J 1	+ 18.75	+ 45.28	+ 53.07	+124.9	+ 78.80	+0.335	+148.5	+ 90.68	+0.302	+172.0	+ 97.68	+0.242
0 1	+ 45.85	+ 45.85	0	+240.9	+128.05	+0.645	+298.4	+157.05	+0.630	+355.9	+174.15	+0.545
J 2	- 40.33	- 40.33	- 47.56	-211.8	-112.53	-0.512	-262.3	-138.13	-0.499	-312.6	-153.03	-0.423
2 3	- 40.33	- 40.33	- 47.56	-211.8	-112.53	-0.512	-262.3	-138.13	-0.499	-312.6	-153.03	-0.423
1 3	+ 28.30	+ 28.30	0	+148.6	+ 78.95	+0.398	+184.1	+ 96.90	+0.389	+219.3	+107.40	+0.336
1 4	+ 28.30	+ 54.84	+ 53.07	+175.1	+105.49	+0.469	+210.6	+123.44	+0.433	+245.8	+133.94	+0.356
3 4	- 11.32	- 11.32	0	- 59.4	- 31.57	-0.160	- 73.6	- 38.72	-0.155	- 87.8	- 42.94	-0.135
4 5	+ 28.30	+ 28.30	0	+148.6	+ 78.95	+0.398	+184.1	+ 96.90	+0.389	+219.3	+107.40	+0.336
5 6	- 20.17	- 20.17	- 47.56	-105.9	- 56.27	-0.230	-131.0	- 68.97	-0.221	-156.2	- 76.52	-0.183
6 7	- 20.17	- 20.17	- 47.56	-105.9	- 56.27	-0.230	-131.0	- 68.97	-0.221	-156.2	- 76.52	-0.183
5 7	+ 28.30	+ 28.30	0	+148.6	+ 78.95	+0.398	+184.1	+ 96.90	+0.389	+219.3	+107.40	+0.336
4 8	+ 33.96	+ 97.64	+127.36	+241.8	+158.44	+0.749	+284.4	+179.94	+0.573	+326.6	+192.44	+0.449
7 9			+212.26			+0.154			+0.145			+0.086
9 10			-103.99			-0.165			-0.156			-0.117
10 11			-103.99			-0.165			-0.156			-0.117
8 9			+212.26	Symmetrical		+0.154	Symmetrical		+0.145	Symmetrical		+0.086
8 11	Symmetrical		- 84.91			-0.062			-0.057			-0.035
8 12			+169.20			+0.347			+0.311			+0.231
11 12			+212.26			+0.154			+0.145			+0.086
11 13			-255.26			-0.273			-0.260			-0.179
13 L			-255.26			-0.273			-0.260			-0.179
12 K			+343.90			+0.249			+0.234			+0.141
12 L			+ 87.56			+0.295			+0.262			+0.201
K L			- 35.80			-0.038			-0.037			-0.013

Memb	
G	J
J	1
G	1
J	2
2	3
1	3
1	4
3	4
4	5
3	6
6	7
5	7
4	8
7	9
7	1
10	11
8	9
8	11
8	1
11	1
11	1
13	L
12	K
12	L
K	L

TABLE XXII.

SPAN 40 FT.

Member	$\frac{PL}{A}$			Column height 16 ft.			Column height 26 ft			Column height 31 ft.		
	A	C	R	$\frac{P_1L}{A}$	$\frac{P_2L}{A}$	$\frac{P_4L}{A}$	$\frac{P_1L}{A}$	$\frac{P_2L}{A}$	$\frac{P_4L}{A}$	$\frac{P_1L}{A}$	$\frac{P_2L}{A}$	$\frac{P_4L}{A}$
				A= +1.667 C= +1.000 R= 0	A= +0.579 C= +1.000 R= 0	A= +0.00414 C= +0.00699 R= -0.00096	A= +3.333 C= +1.000 R= 0	A= +1.422 C= +1.000 R= 0	A= +0.00610 C= +0.00426 R= -0.00087	A= +4.167 C= +1.000 R= 0	A= +1.700 C= +1.000 R= 0	A= +0.00563 C= +0.00325 R= -0.00089
G J	- 9.547	- 9.547	- 9.945	- 25.0	- 15.08	-0.098	- 31.8	- 23.15	-0.090	- 39.8	- 21.32	-0.063
J 1	+32.547	+67.924	+ 70.755	+122.1	+ 86.79	+0.542	+ 108.5	+ 114.12	+0.427	+ 135.6	+123.32	+0.341
G 1	+67.983	+67.983	0	+181.2	+107.38	+0.757	+ 226.3	+ 164.58	+0.704	+ 283.3	+183.58	+0.604
J 2	-54.960	-54.960	- 57.251	-146.6	- 86.84	-0.557	- 183.2	- 133.11	-0.519	- 228.7	-148.36	-0.438
2 3	-54.960	-54.960	- 57.251	-146.6	- 86.84	-0.557	- 183.2	- 133.11	-0.519	- 228.7	-148.36	-0.438
1 3	+42.453	+42.453	0	+113.2	+ 67.05	+0.473	+ 141.4	+ 102.80	+0.440	+ 176.8	+114.60	+0.377
1 4	+42.453	+77.830	+ 70.755	+148.5	+102.43	+0.652	+ 141.4	+ 138.18	+0.530	+ 176.8	+149.98	+0.429
3 4	-16.982	-16.982	0	- 45.3	- 26.82	+0.189	- 56.6	- 41.13	-0.176	- 70.7	- 45.83	-0.151
4 5	+42.453	+42.453	0	+113.2	+ 67.05	+0.473	+ 141.4	+ 102.80	+0.440	+ 176.8	+114.60	+0.377
5 6	-27.480	-27.480	- 57.251	- 73.3	- 43.42	-0.251	- 91.6	- 66.53	-0.235	- 114.6	- 74.18	-0.193
6 7	-27.480	-27.480	- 57.251	- 73.3	- 43.42	-0.251	- 91.6	- 66.53	-0.235	- 114.6	- 74.18	-0.193
5 7	+42.453	+42.453	0	+113.2	+ 67.05	+0.473	+ 141.4	+ 102.80	+0.440	+ 176.8	+114.60	+0.377
4 8	+50.943	+135.849	+169.811	+220.8	+165.40	+0.999	+ 169.8	+ 208.30	+0.741	+ 212.2	+222.45	+0.573
7 9			+283.019			+0.203			+0.194			+0.124
7 10			-125.952			-0.186			-0.176			-0.132
10 11			-125.952			-0.186			-0.176			-0.132
8 9	Symmetrical		+283.019	Symmetrical		+0.203	Symmetrical		+0.194	Symmetrical		+0.124
8 11			-113.204			-0.081			-0.078			-0.050
8 12			+212.264			+0.517			+0.406			+0.303
11 12			+283.019			+0.203			+0.194			+0.124
11 13			-309.152			-0.316			-0.300			-0.213
13 L			-309.152			-0.316			-0.300			-0.213
12 K			+453.216			+0.324			+0.310			+0.199
12 L			+146.225			+0.470			+0.361			+0.274
K L			- 53.702			+0.056			-0.052			-0.030

Member

G	J
J	1
G	1
J	2
2	3
1	3
1	4
3	4
4	5
3	6
6	7
5	7
4	8
7	9
7	10
10	11
8	9
8	11
8	12
11	12
11	13
13	L
12	K
12	L
K	L

TABLE XXIII.

SPAN 50 FT.

Member	$\frac{PL}{A}$			Column height 16 ft.			Column height 26 ft.			Column height 31 ft.		
				$\frac{P_1L}{A}$	$\frac{P_2L}{A}$	$\frac{P_4L}{A}$	$\frac{P_1L}{A}$	$\frac{P_2L}{A}$	$\frac{P_4L}{A}$	$\frac{P_1L}{A}$	$\frac{P_2L}{A}$	$\frac{P_4L}{A}$
	A	C	R	A= +1.667 C= +1.000 R= 0	A= +0.614 C= +1.000 R= 0	A= +0.00457 C= +0.00736 R= -0.00074	A= +3.333 C= +1.000 R= 0	A= +1.451 C= +1.000 R= 0	A= +0.00635 C= +0.00436 R= -0.00069	A= +4.167 C= +1.000 R= 0	A= +1.770 C= +1.000 R= 0	A= +0.00602 C= +0.00337 R= -0.00070
G J	-7.64	-7.64	-9.94	-20.3	-12.33	-0.078	-33.0	-18.74	-0.075	-32.6	-17.49	-0.053
J 1	+21.11	+60.50	+78.78	+95.7	+73.47	+0.482	+130.9	+91.10	+0.344	+148.4	+97.85	+0.288
G 1	+62.63	+62.63	0	+166.9	+101.13	+0.748	+221.2	+153.38	+0.671	+323.6	+173.43	+0.588
J 2	-40.45	-40.45	-52.66	-107.9	-65.30	-0.444	-175.3	-99.15	-0.397	-209.1	-112.05	-0.343
2 3	-40.45	-40.45	-52.66	-107.9	-65.30	-0.444	-175.3	-99.15	-0.397	-209.1	-112.05	-0.343
1 3	+42.45	+42.45	0	+113.3	+68.50	+0.506	+184.1	+103.95	+0.455	+219.3	+117.60	+0.399
1 4	+37.82	+77.21	+78.78	+140.2	+100.46	+0.683	+203.2	+132.11	+0.522	+234.9	+144.21	+0.445
3 4	-16.98	-16.98	0	-45.3	-27.40	-0.203	-73.6	-41.58	-0.182	-87.7	-47.03	-0.159
4 5	+42.45	+42.45	0	+113.3	+68.50	+0.506	+184.1	+103.95	+0.455	+219.3	+117.60	+0.399
3 6	-20.22	-20.22	-52.66	-53.9	-32.65	-0.202	-87.6	-49.57	-0.180	-104.5	-56.02	-0.153
6 7	-20.22	-20.22	-52.66	-53.9	-32.65	-0.202	-87.6	-49.57	-0.180	-104.5	-56.02	-0.153
5 7	+42.45	+42.45	0	+113.3	+68.50	+0.506	+184.1	+103.95	+0.455	+219.3	+117.60	+0.399
4 8	+50.94	+157.08	+212.26	+242.0	+188.38	+1.233	+326.9	+230.98	+0.853	+369.3	+247.28	+0.687
7 9			+353.77			+0.243			+0.213			+0.150
7 10			-115.87			-0.155			-0.137			-0.109
10 11			-115.87			-0.155			-0.137			-0.109
8 9	Symmetrical		+353.77	Symmetrical		+0.243	Symmetrical		+0.213	Symmetrical		+0.150
8 11			-141.50			-0.098			-0.085			-0.059
8 12			+236.34			+0.566			+0.414			+0.322
11 12			+353.77			+0.243			+0.213			+0.150
11 13			-284.42			-0.271			-0.238			-0.180
13 L			-284.42			-0.271			-0.238			-0.180
12 K			+512.88			+0.366			+0.319			+0.226
12 L			+97.17			+0.469			+0.331			+0.263
K L			-53.70			-0.045			-0.045			-0.028

Membe	
G	J
J	1
G	1
J	2
2	3
1	3
1	4
3	4
4	5
3	6
6	7
5	7
4	8
7	9
7	1
10	1
8	9
8	1
8	1
11	1
11	1
13	L
12	K
12	L
K	L

TABLE XXIV.

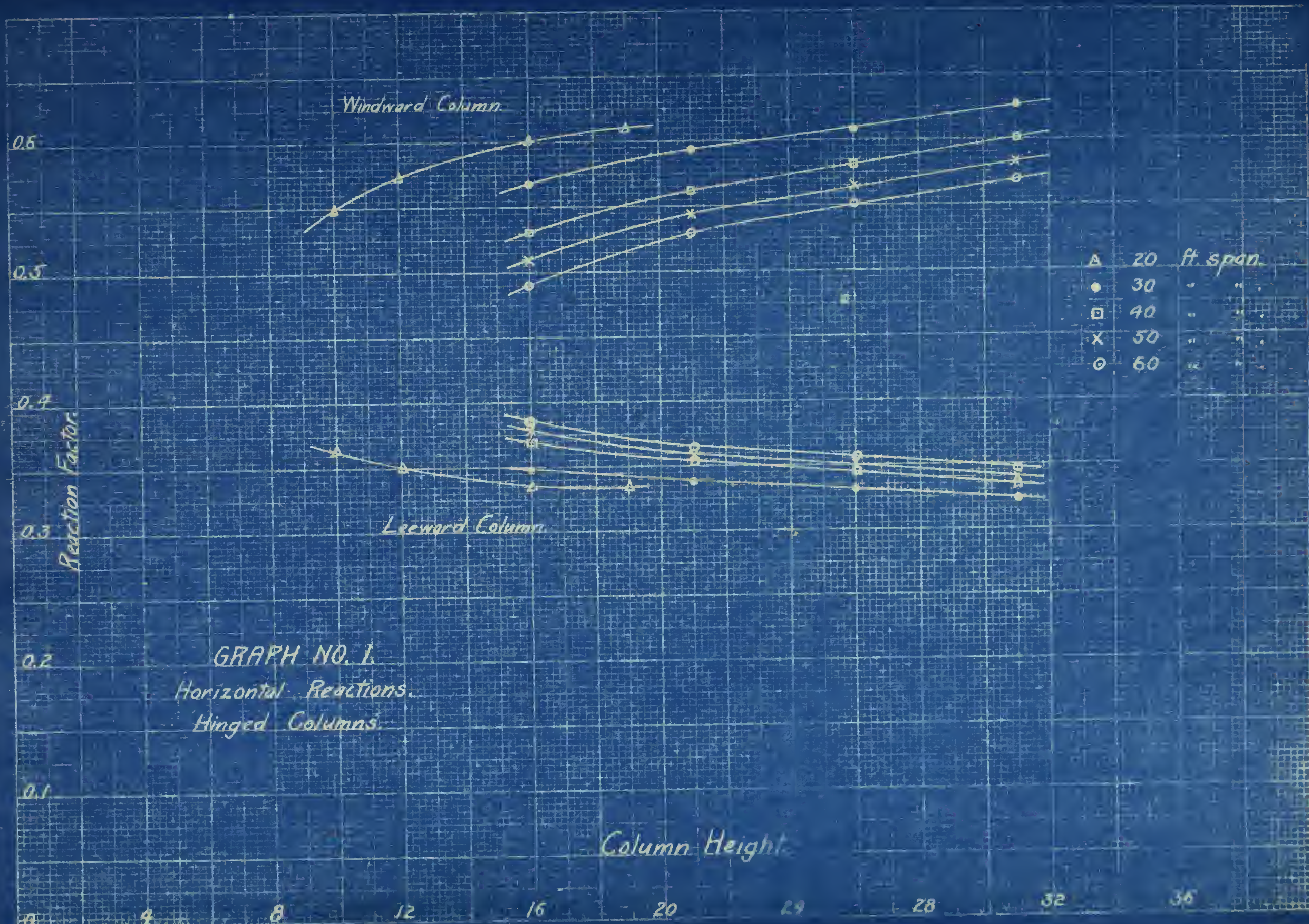
SPAN 60 FT.

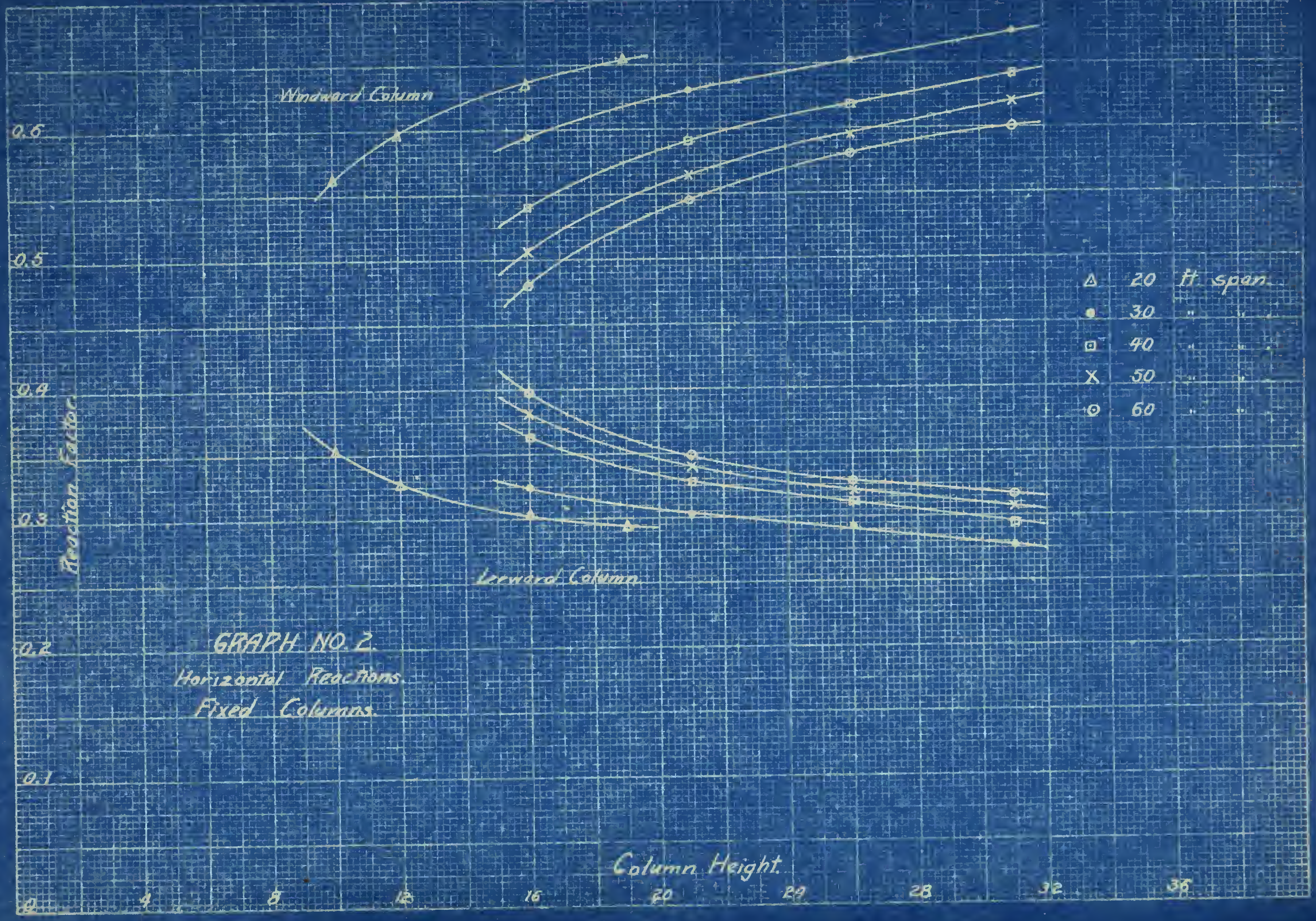
Member	PL/A (Span 60 ft.)			Column height 16 ft.			Column height 26 ft.			Column height 31 ft.		
				$\frac{P_1L}{A}$	$\frac{P_2L}{A}$	$\frac{P_4L}{A}$	$\frac{P_1L}{A}$	$\frac{P_2L}{A}$	$\frac{P_4L}{A}$	$\frac{P_1L}{A}$	$\frac{P_2L}{A}$	$\frac{P_4L}{A}$
	A	C	R	A = +1.667 C = +1.000 R = 0	A = +0.630 C = +1.000 R = 0	A = +0.00479 C = +0.00755 R = -0.00061	A = +3.333 C = +1.000 R = 0	A = +1.464 C = +1.000 R = 0	A = +0.00646 C = +0.00440 R = -0.00057	A = +4.167 C = +1.000 R = 0	A = +1.801 C = +1.000 R = 0	A = +0.00622 C = +0.00343 R = -0.00059
G J	- 6.36	- 6.36	- 9.94	- 17.0	- 10.37	-0.072	- 27.6	- 15.68	-0.063	- 27.2	- 14.73	-0.046
J 1	+ 12.02	+ 54.96	+ 85.88	+ 75.0	+ 62.53	+0.421	+ 95.0	+ 72.56	+0.271	+105.0	+ 76.60	+0.213
G 1	+ 60.53	+ 60.53	0	+161.4	+ 98.71	+0.747	+262.3	+149.23	+0.657	+312.5	+169.63	+0.584
J 2	- 37.31	- 37.31	- 58.29	- 99.4	- 60.81	-0.426	-161.6	- 91.91	-0.372	-192.7	-104.46	-0.326
2 3	- 37.31	- 37.31	- 58.29	- 99.4	- 60.81	-0.426	-161.6	- 91.91	-0.372	-192.7	-104.46	-0.326
1 3	+ 42.45	+ 42.45	0	+113.2	+ 69.20	+0.525	+183.9	+104.65	+0.461	+219.5	+118.90	+0.410
1 4	+ 34.35	+ 77.29	+ 85.88	+134.5	+ 98.94	+0.697	+191.7	+127.59	+0.513	+220.5	+139.09	+0.429
3 4	- 16.98	- 16.98	0	- 45.3	- 27.68	-0.209	- 73.6	- 41.83	-0.185	- 87.8	- 47.56	-0.164
4 5	+ 37.82	+ 37.82	0	+100.8	+ 61.64	+0.467	+163.8	+ 93.22	+0.410	+195.5	+105.92	+0.365
3 6	- 18.65	- 18.65	- 58.29	- 49.8	- 30.41	-0.195	- 80.9	- 45.95	-0.169	- 96.4	- 52.25	-0.146
6 7	- 18.65	- 18.65	- 58.29	- 49.8	- 30.41	-0.195	- 80.9	- 45.95	-0.169	- 96.4	- 52.25	-0.146
5 7	+ 37.82	+ 37.82	0	+100.8	+ 61.64	+0.467	+163.8	+ 93.22	+0.410	+195.5	+105.92	+0.365
4 8	+ 50.94	+178.30	+254.72	+263.2	+210.40	+1.437	+348.1	+252.90	+0.968	+390.5	+270.90	+0.781
7 9			+378.15			+0.236			+0.185			+0.145
7 10			-128.24			-0.152			-0.129			-0.106
10 11			-128.24			-0.152			-0.129			-0.106
8 9	Symmetrical		+378.15	Symmetrical		+0.236	Symmetrical		+0.185	Symmetrical		+0.145
8 11			-169.80			-0.106			-0.088			-0.065
8 12			+257.63			+0.592			+0.416			+0.329
11 12			+424.53			+0.266			+0.220			+0.163
11 13			-314.77			-0.269			-0.226			-0.177
13 L			-314.77			-0.269			-0.226			-0.177
12 K			+605.30			+0.378			+0.313			+0.232
12 L			+ 34.35			+0.452			+0.300			+0.240
K L			- 53.70			-0.046			-0.038			-0.025

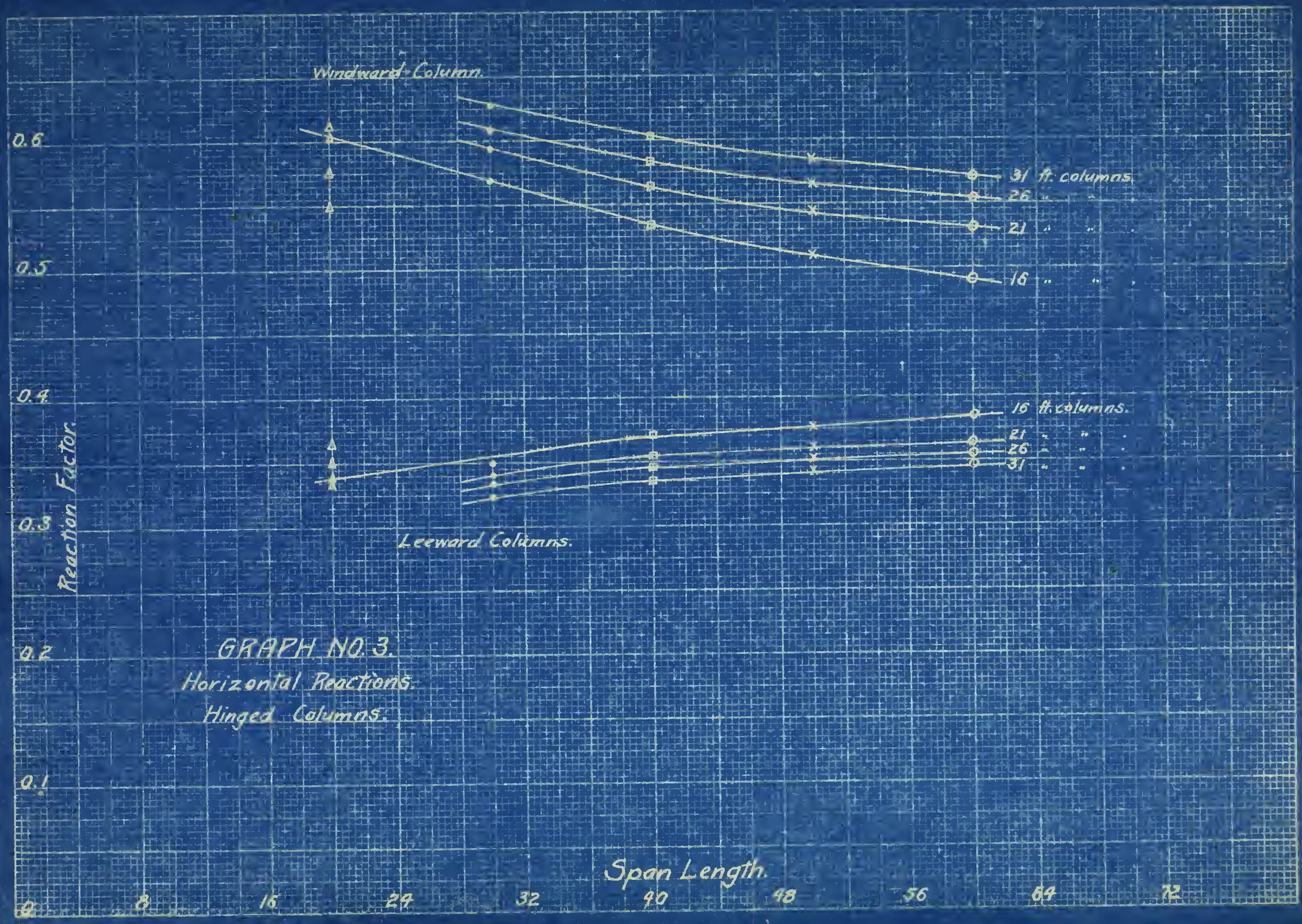
Span length l
20
20
20
20
30
30
30
30
40
40
40
40
50
50
50
50
60
60
60
60

TABLE XXV.

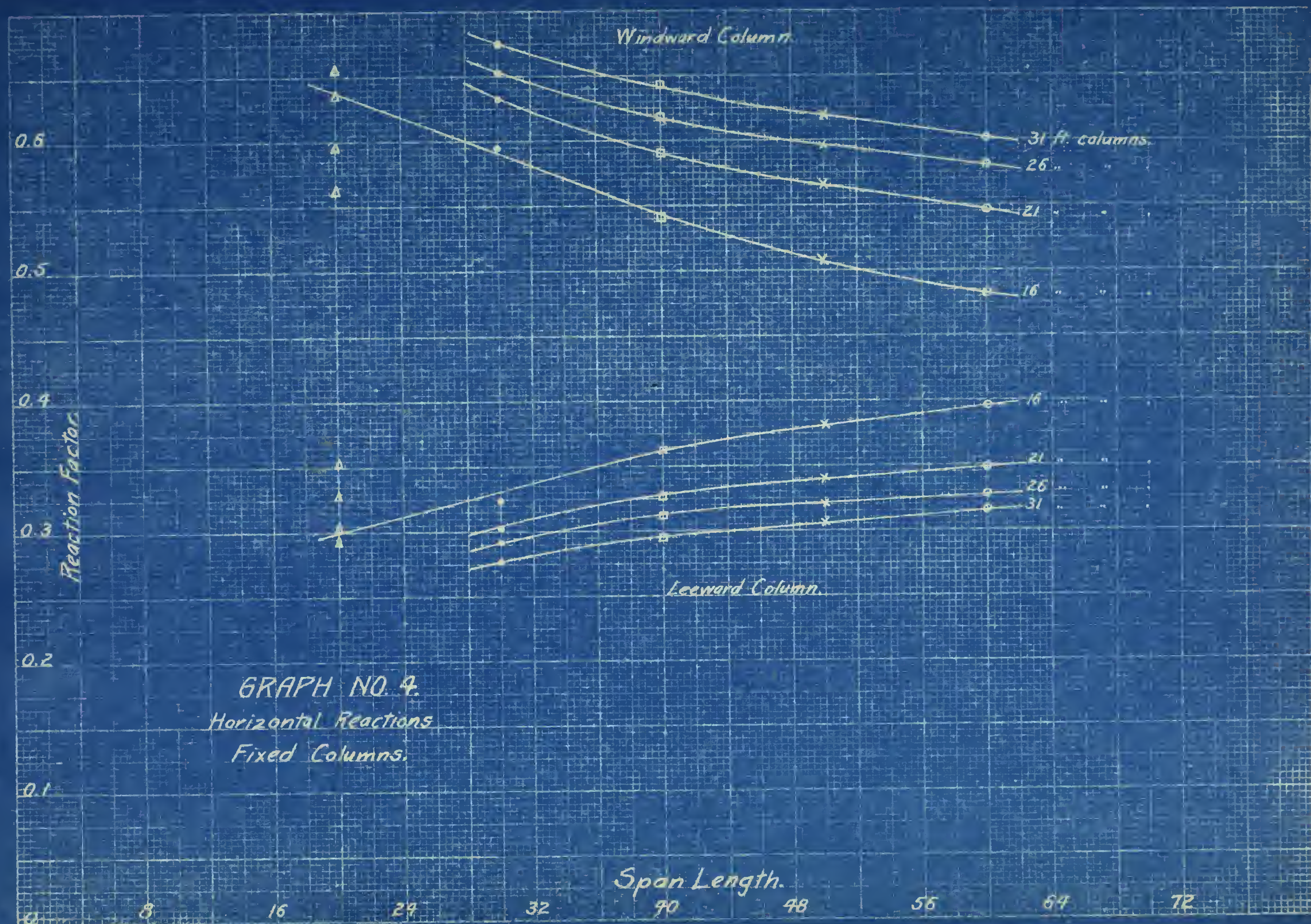
Span length l	Column height h	Ratio h/l	Total height t	Class I					Class II								
				Case A.			Case B.		Case A			Case B		Case C			
				H_L/t	H_R/t	H_R/H	V_L/t	V_R/t	H_L/t	H_R/t	H_R/H	V_L/t	V_R/t	M_L/t	M_R/t	Y_L/d	Y_R/d
20	10	.500	15	.550	.365	.399	.054	.443	.563	.352	.385	.194	.303	17.102	16.554	.518	.654
20	12	.600	17	.575	.351	.379	-.039	.478	.598	.327	.354	.138	.300	22.528	19.484	.486	.620
20	16	.800	21	.603	.336	.358	-.201	.556	.636	.303	.323	.039	.316	31.986	25.531	.435	.585
20	19	.950	24	.610	.337	.356	-.308	.619	.654	.293	.309	-.014	.324	40.779	29.964	.439	.569
30	16	.533	23.5	.569	.349	.380	.0185	.457	.595	.324	.353	.186	.290	32.639	27.662	.481	.593
30	21	.700	28.5	.594	.339	.364	-.126	.518	.631	.302	.324	.095	.297	44.405	35.140	.435	.570
30	26	.867	33.5	.609	.334	.354	-.252	.586	.652	.292	.309	.027	.307	58.219	42.072	.432	.547
30	31	1.033	38.5	.628	.323	.340	-.367	.657	.675	.276	.290	-.022	.312	74.078	50.177	.439	.562
40	16	.400	26	.532	.370	.410	.157	.416	.540	.362	.401	.278	.296	31.29	26.58	.621	.612
40	21	.525	31	.564	.354	.385	.027	.454	.591	.327	.356	.184	.296	41.51	34.14	.488	.581
40	26	.650	36	.583	.346	.373	-.084	.499	.618	.311	.335	.117	.297	55.06	41.84	.471	.560
40	31	.775	41	.602	.335	.358	-.184	.548	.643	.295	.314	.064	.299	68.97	49.77	.457	.568
50	16	.320	28.5	.511	.378	.425	.254	.400	.508	.381	.429	.350	.304	30.11	27.46	.634	.601
50	21	.420	33.5	.546	.360	.397	.133	.423	.566	.339	.375	.259	.297	40.74	34.87	.498	.571
50	26	.520	38.5	.567	.351	.382	.032	.453	.597	.320	.349	.191	.293	52.98	42.71	.465	.555
50	31	.620	43.5	.584	.342	.369	-0.060	.488	.620	.307	.332	.137	.292	66.32	51.47	.452	.558
60	16	.267	31	.492	.385	.439	.331	.390	.480	.397	.453	.411	.311	29.00	28.26	.643	.593
60	21	.350	36	.530	.363	.407	.216	.405	.547	.347	.388	.321	.300	39.38	35.84	.494	.574
60	26	.433	41	.553	.354	.390	.119	.426	.580	.327	.361	.250	.295	50.79	43.38	.454	.558
60	31	.517	46	.570	.347	.379	.034	.452	.601	.316	.345	.194	.292	62.41	52.49	.429	.554


Windward Column
Leeward Column
Δ 20 ft. span.
● 30 " "
⊡ 40 " "
X 50 " "
⊙ 60 " "
Reaction Factor.
0.6
0.5
0.4
0.3
0.2
0.1
0
4
8
12
16
20
24
28
32
36
Column Height
GRAPH NO. 1.
Horizontal Reactions.
Hinged Columns.

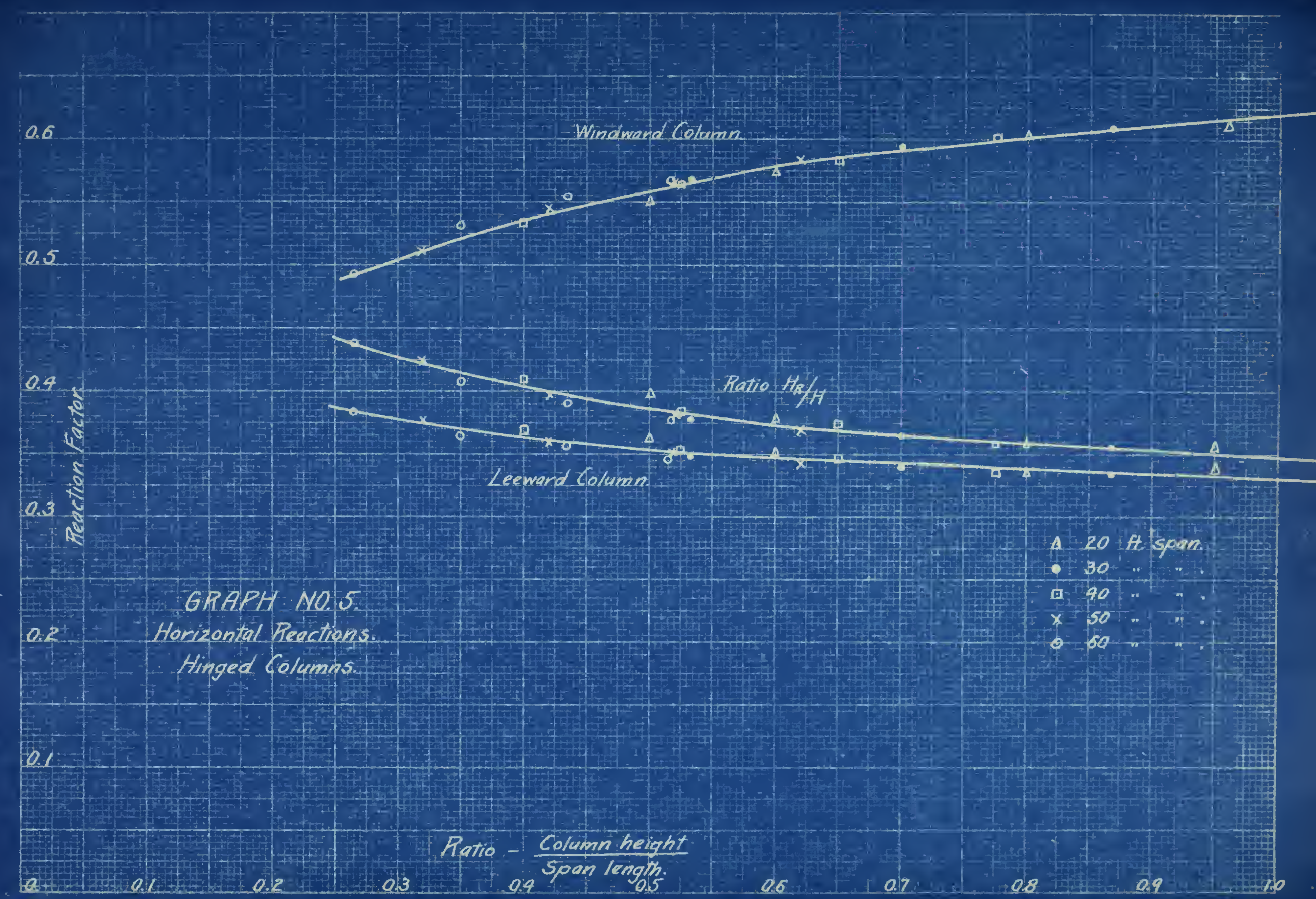

GRAPH NO. 2.
Horizontal Reactions.
Fixed Columns.
Windward Column
Leeward Column
Reaction Factor
Column Height.
0.6
0.5
0.4
0.3
0.2
0.1
0
4
8
12
16
20
24
28
32
36
20 ft. span.
30
40
50
60

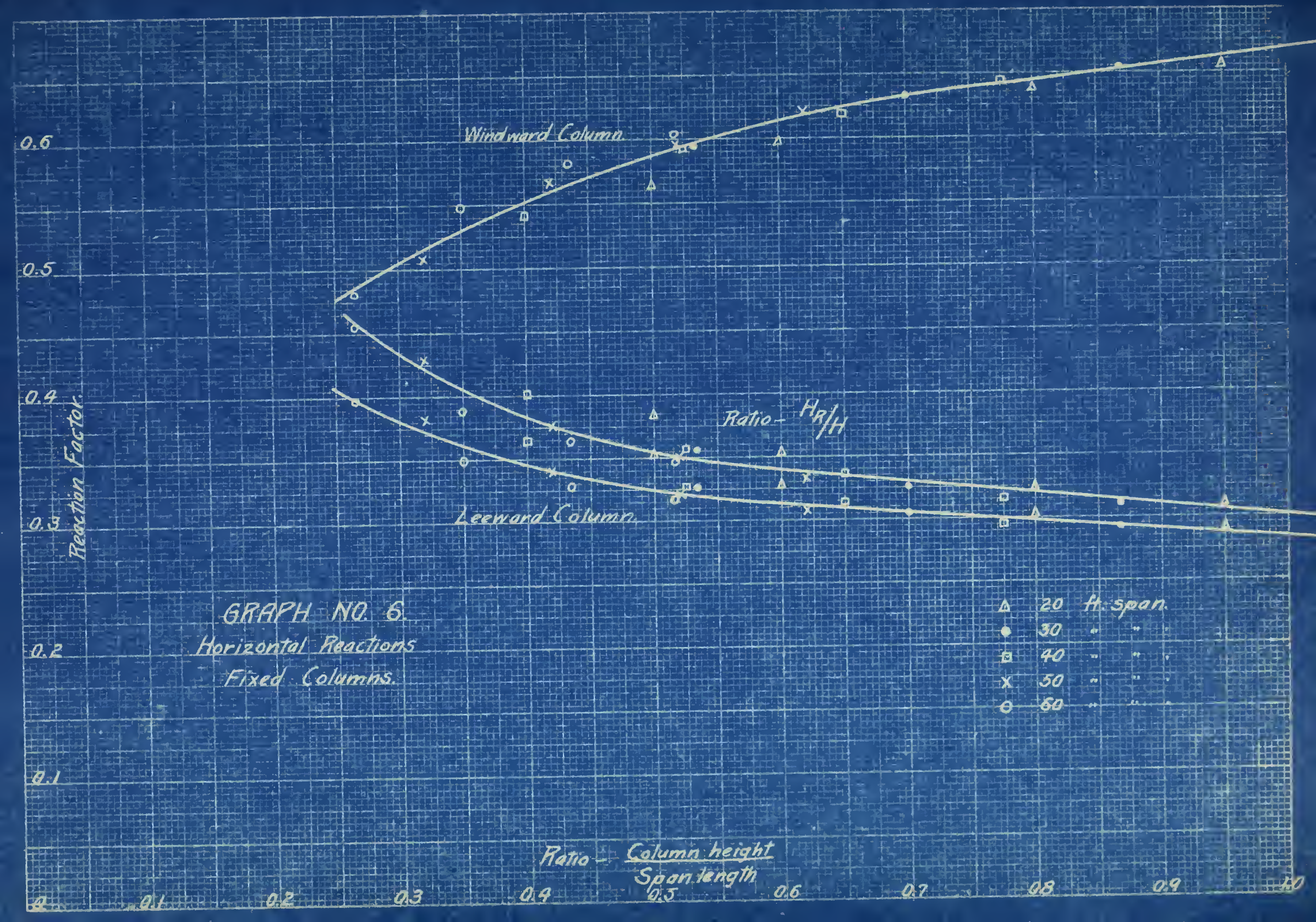


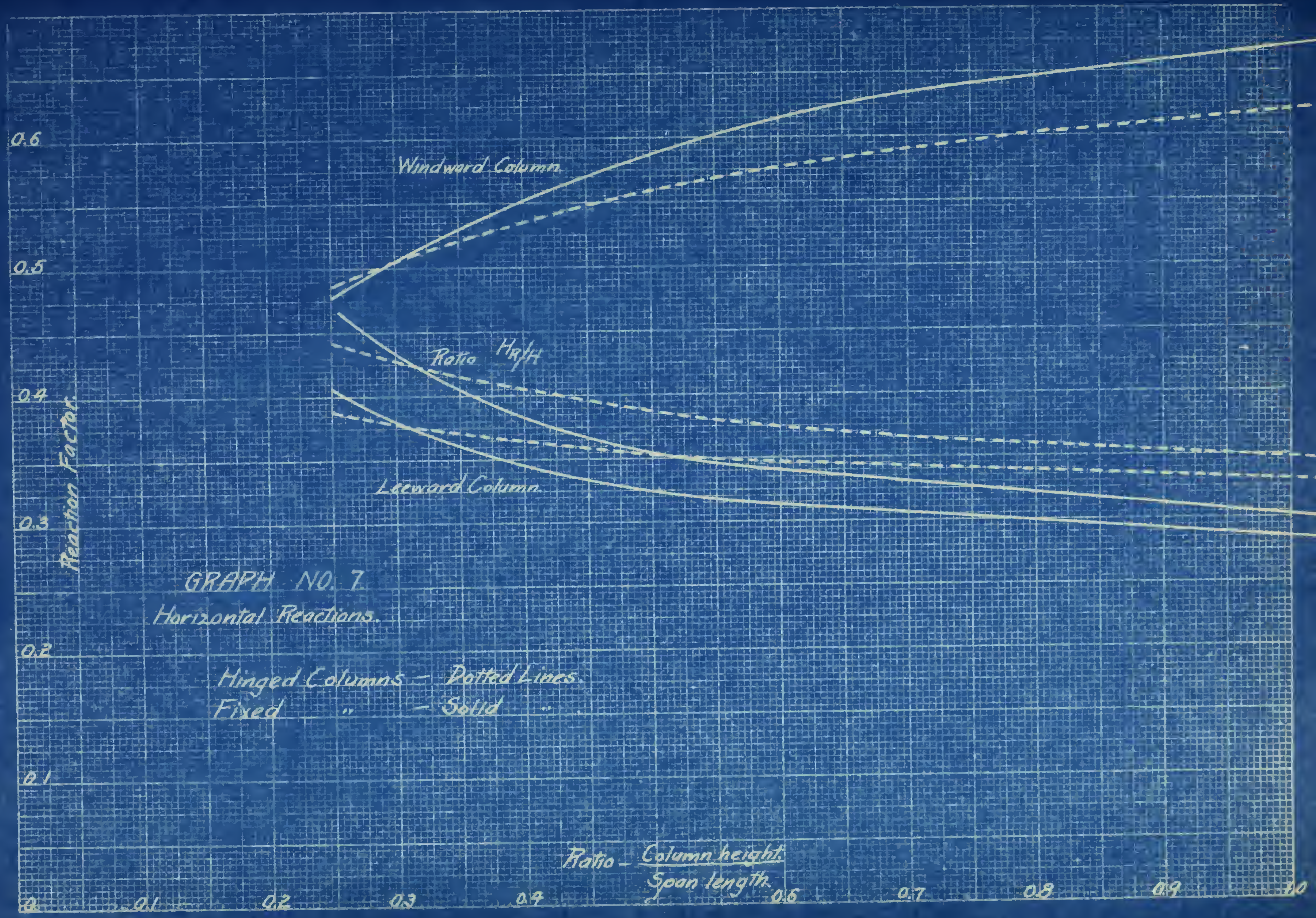
GRAPH NO. 3.
Horizontal Reactions.
Hinged Columns.
Windward Column.
Leeward Columns.
31 ft. columns.
26 " "
21 " "
16 " "
16 ft. columns.
21 " "
26 " "
31 " "
Reaction Factor.
Span Length.
0
0.1
0.2
0.3
0.4
0.5
0.6
8
16
24
32
40
48
56
64
72

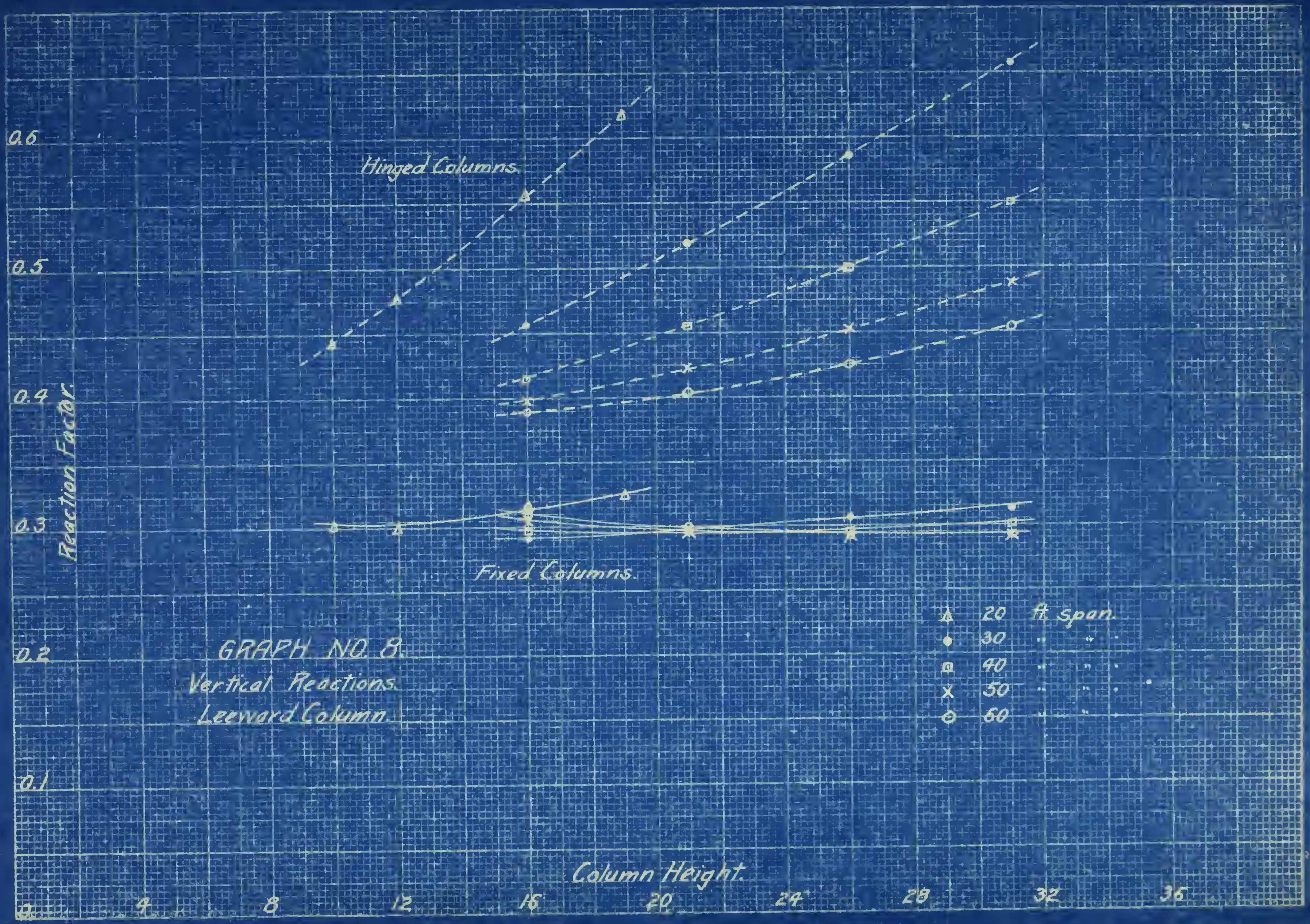


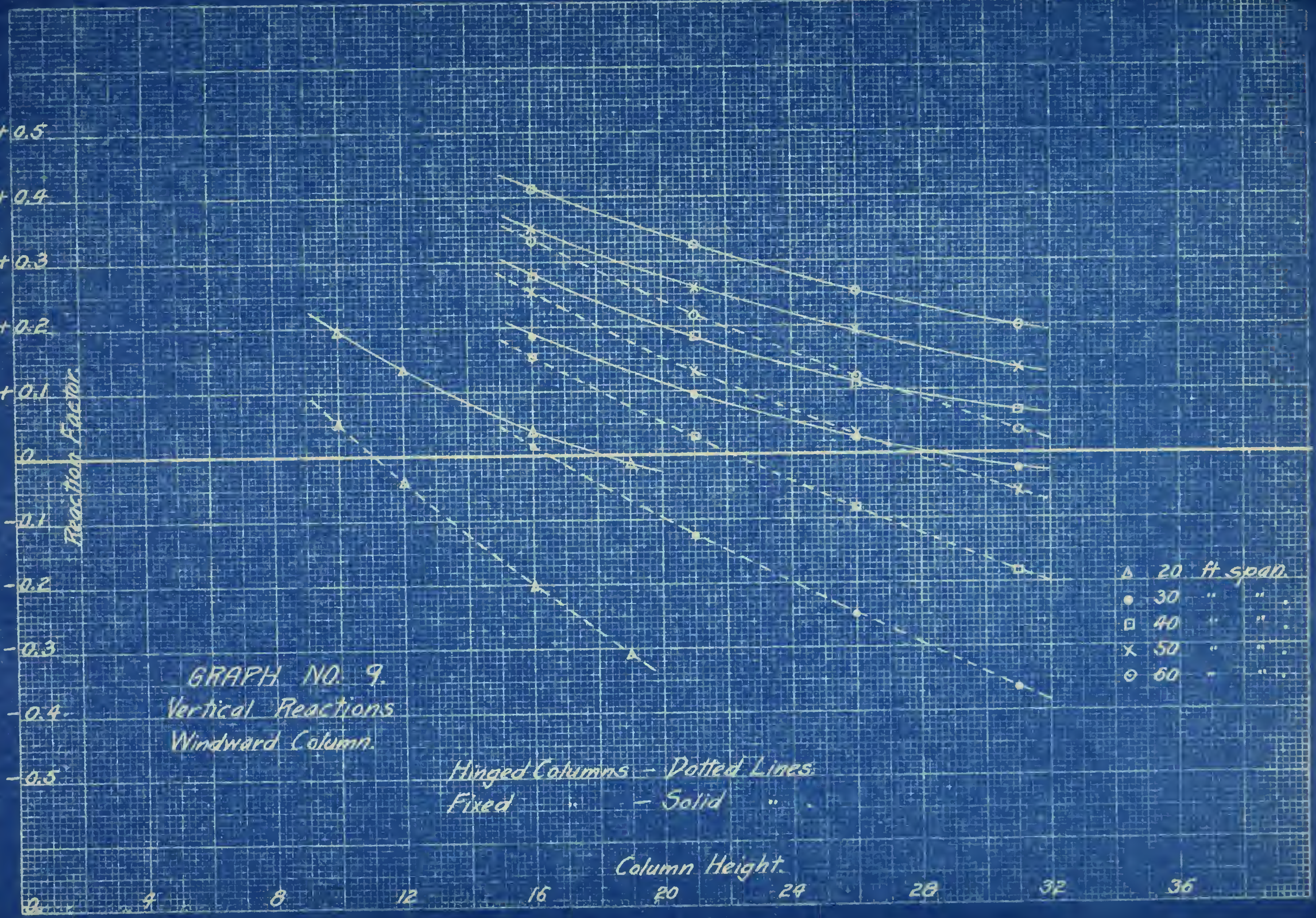
Windward Column.
31 ft. columns.
26 " "
21 " "
16 " "
16 " "
21 " "
26 " "
31 " "
Leeward Column.
Reaction Factor.
0.6
0.5
0.4
0.3
0.2
0.1
0
8
16
24
32
40
48
56
64
72
Span Length.
GRAPH NO. 4.
Horizontal Reactions
Fixed Columns.


Windward Column
Ratio H_R/H
Leeward Column
Reaction Factor
GRAPH NO. 5.
Horizontal Reactions.
Hinged Columns.
Ratio - Column height / Span length.
20 ft. span.
30 " "
40 " "
50 " "
60 " "
0.6
0.5
0.4
0.3
0.2
0.1
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1.0

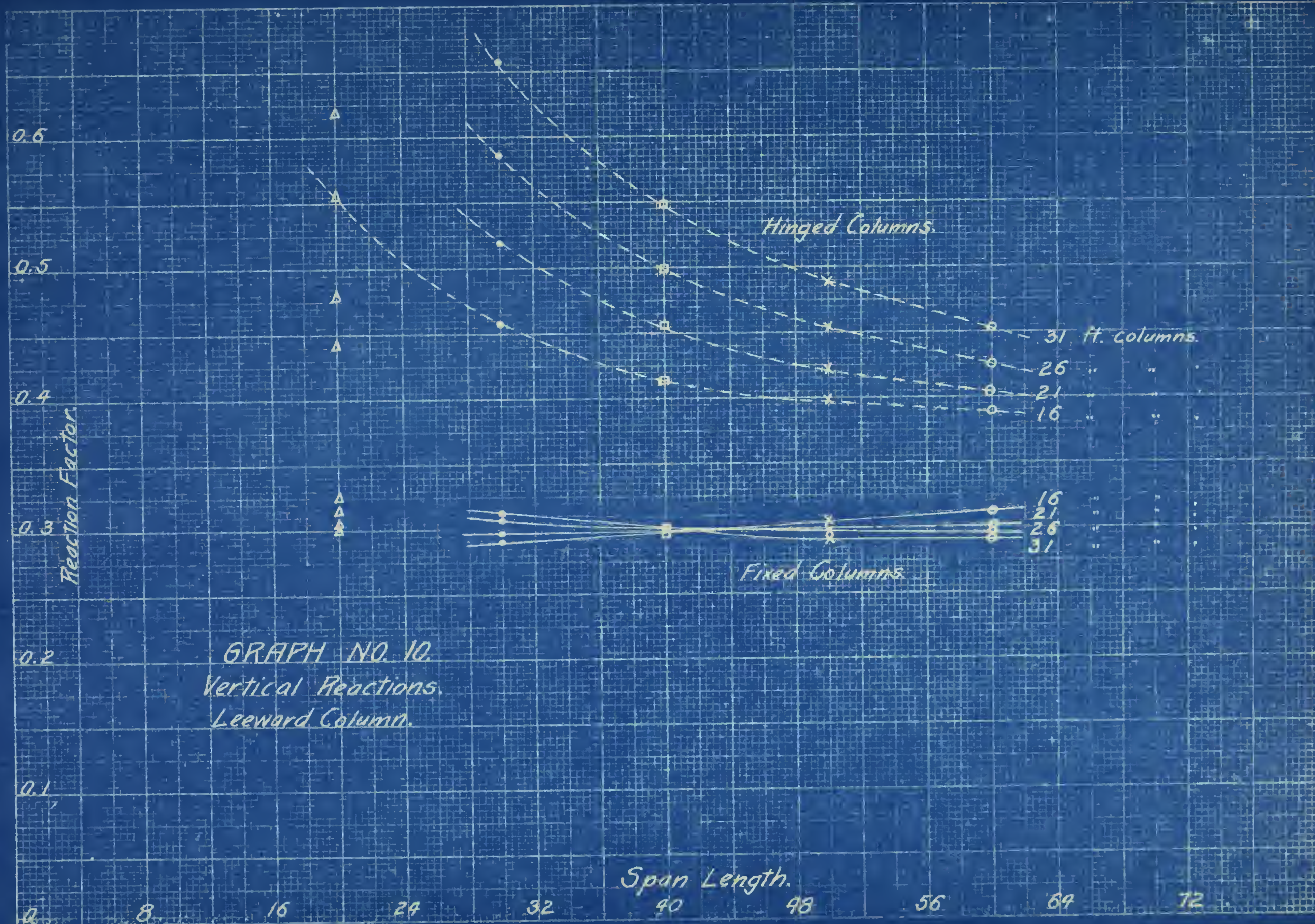

GRAPH NO. 6.
Horizontal Reactions
Fixed Columns.
Windward Column
Leeward Column
Ratio - H_R/H
Reaction Factor
Ratio - $\frac{\text{Column height}}{\text{Span length}}$
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1.0
20 ft. span.
30 " "
40 " "
50 " "
60 " "


GRAPH NO. 7
Horizontal Reactions.
Hinged Columns - Dotted Lines.
Fixed " - Solid "
Windward Column
Ratio H_R/H
Leeward Column
Reaction Factor.
Ratio - Column height / Span length.
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1.0

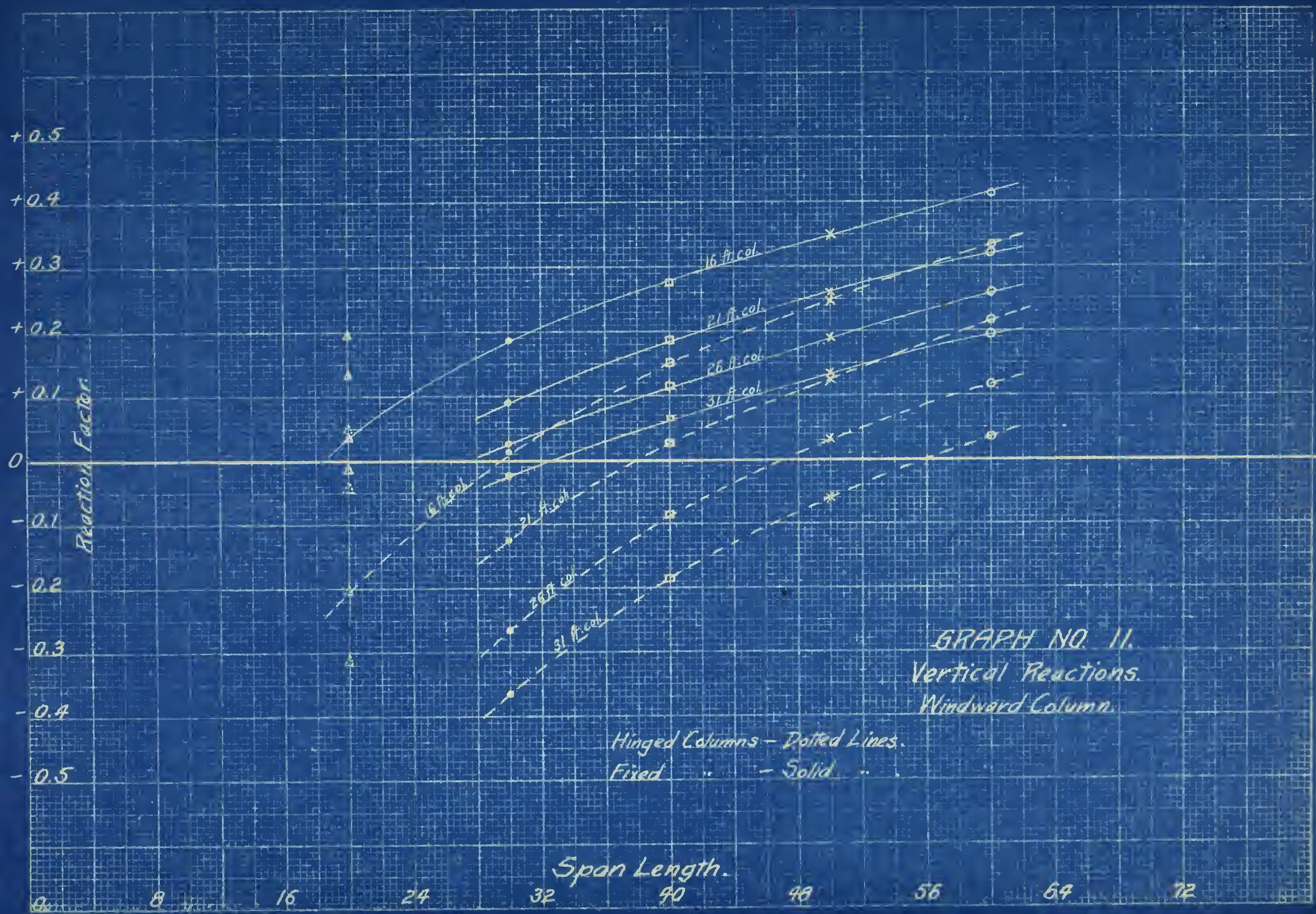

GRAPH NO. 8.
Vertical Reactions.
Leeward Column.
Hinged Columns.
Fixed Columns.
Reaction Factor.
Column Height.
0.6
0.5
0.4
0.3
0.2
0.1
0
4
8
12
16
20
24
28
32
36
20 ft. span.
30 " "
40 " "
50 " "
60 " "

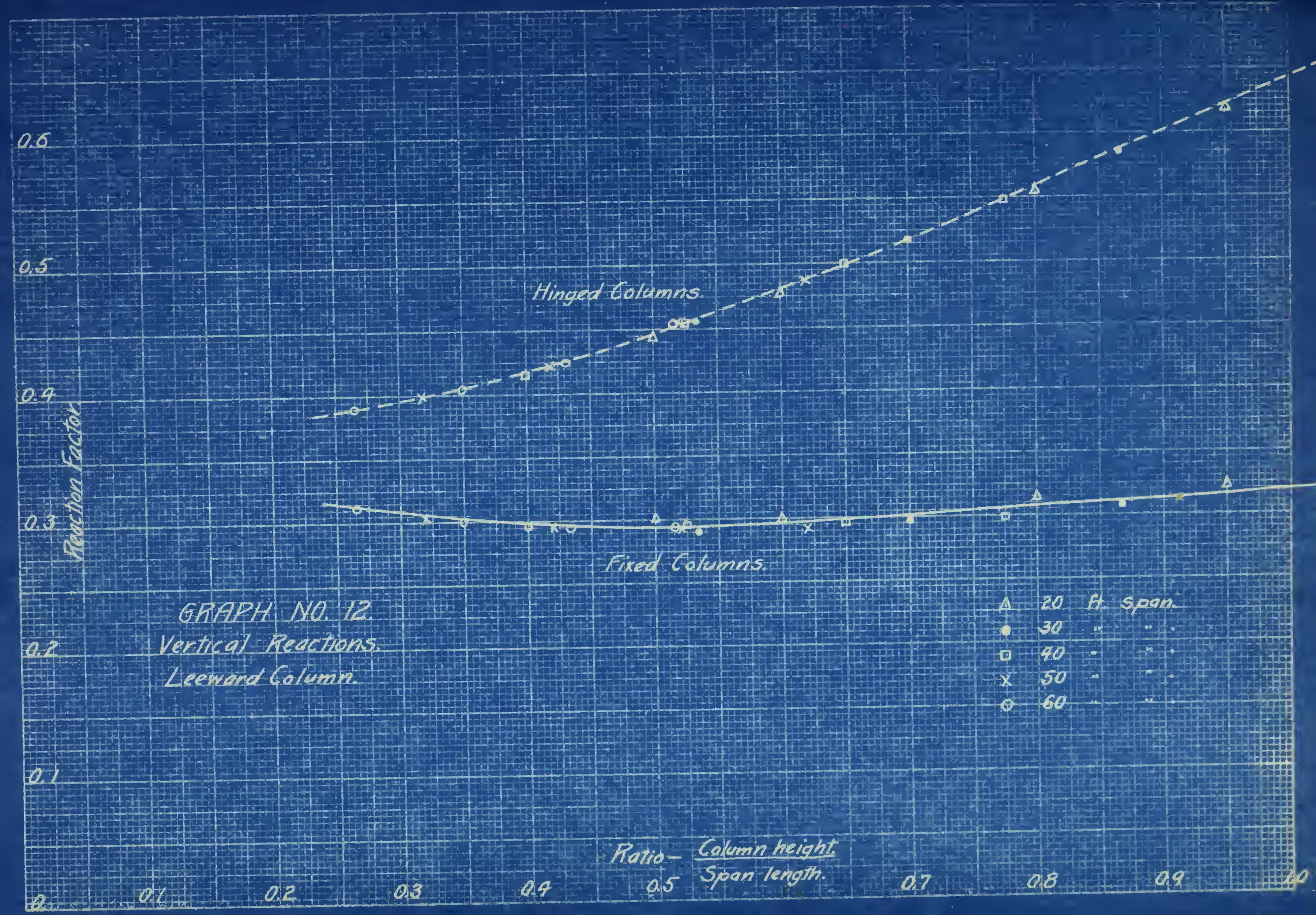


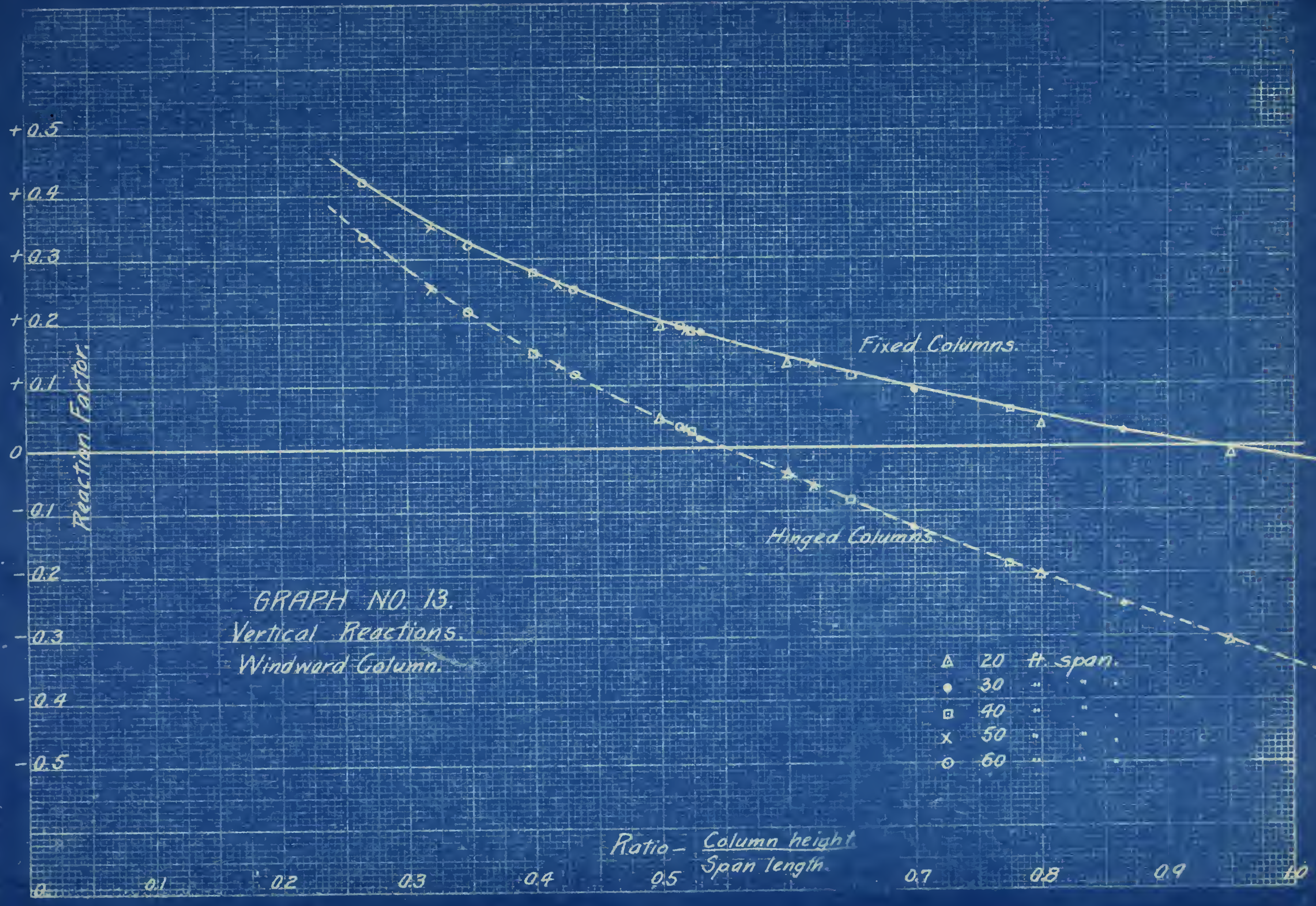
GRAPH NO. 9.
Vertical Reactions
Windward Column.
Hinged Columns - Dotted Lines.
Fixed " - Solid "
Column Height.
Reaction Factor
+0.5
+0.4
+0.3
+0.2
+0.1
0
-0.1
-0.2
-0.3
-0.4
-0.5
0
4
8
12
16
20
24
28
32
36
△ 20 ft. span.
● 30 " "
□ 40 " "
× 50 " "
⊙ 60 " "

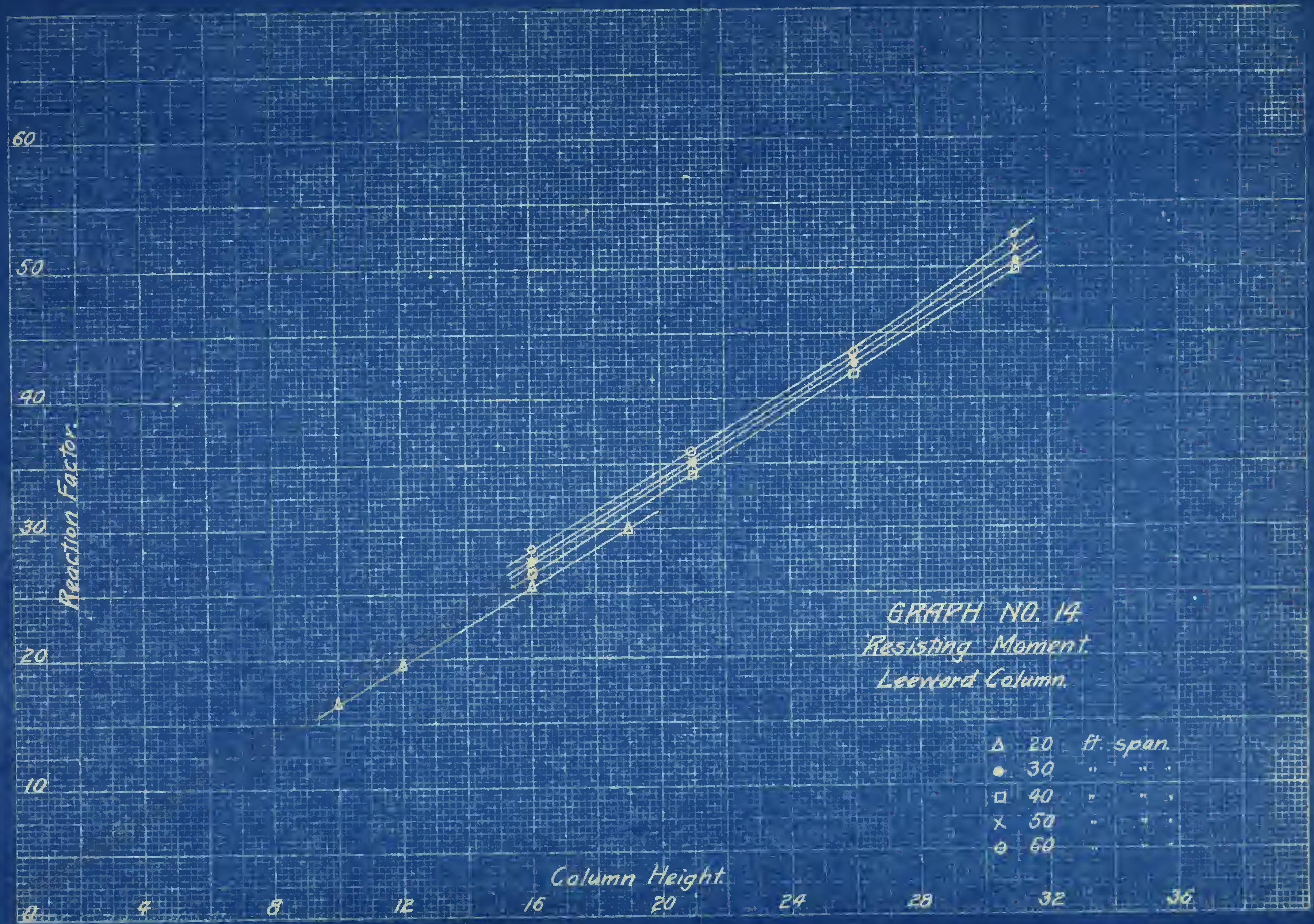


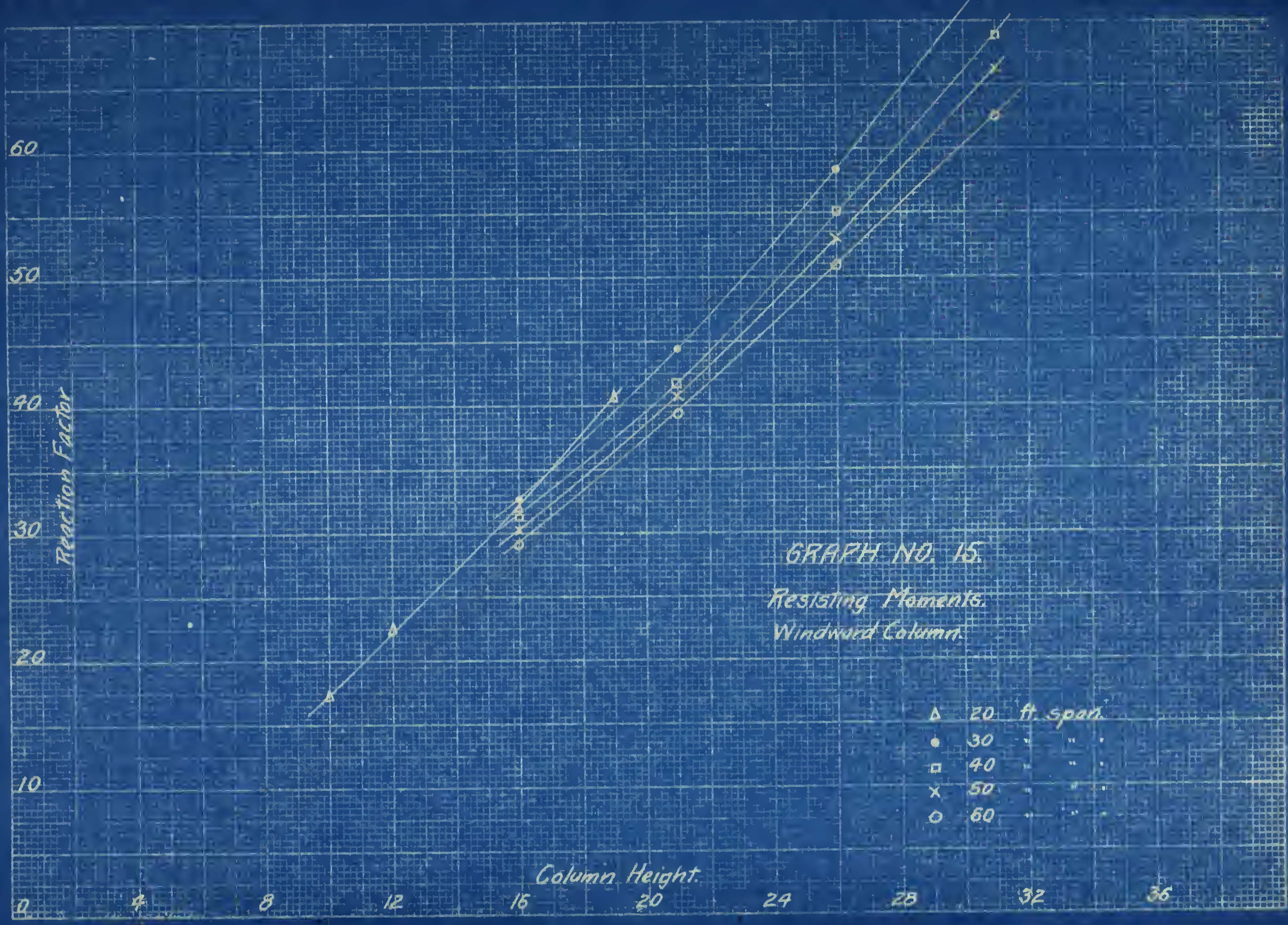
GRAPH NO. 10.
Vertical Reactions.
Leeward Column.
Hinged Columns.
Fixed Columns.
31 ft. columns.
26 " "
21 " "
16 " "
16
21
26
31
Reaction Factor.
0.6
0.5
0.4
0.3
0.2
0.1
0
8
16
24
32
40
48
56
64
72
Span Length.

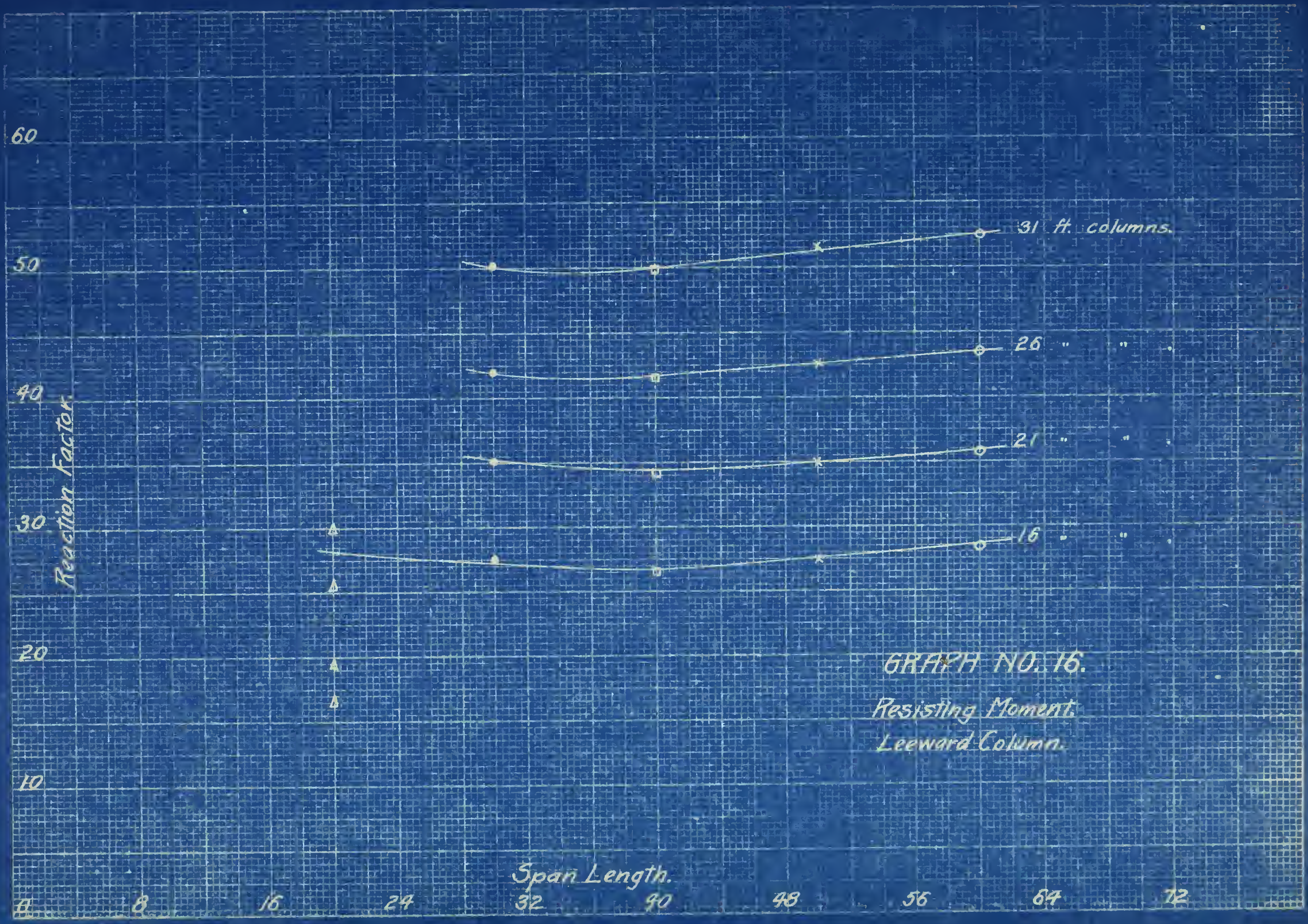

GRAPH NO. 11.
Vertical Reactions.
Windward Column.
Hinged Columns – Dotted Lines.
Fixed " – Solid "
16 ft. col.
21 ft. col.
26 ft. col.
31 ft. col.
16 ft. col.
21 ft. col.
26 ft. col.
31 ft. col.
Reaction Factor.
+0.5
+0.4
+0.3
+0.2
+0.1
0
-0.1
-0.2
-0.3
-0.4
-0.5
Span Length.
0
8
16
24
32
40
48
56
64
72

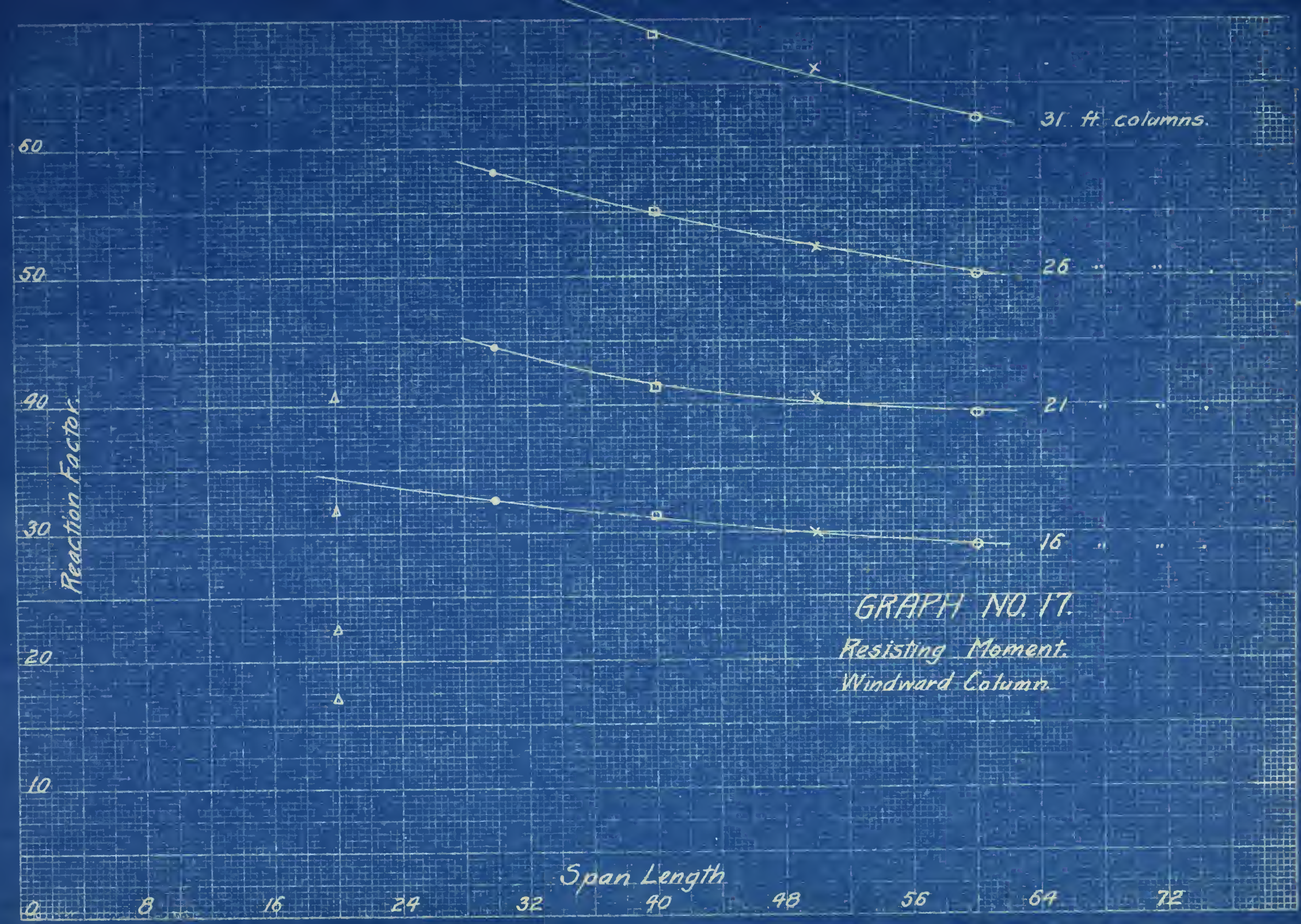

GRAPH NO. 12.
Vertical Reactions.
Leeward Column.
Hinged Columns.
Fixed Columns.
Reaction Factor
Ratio – Column height / Span length.
20 ft. span.
30
40
50
60
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1.0

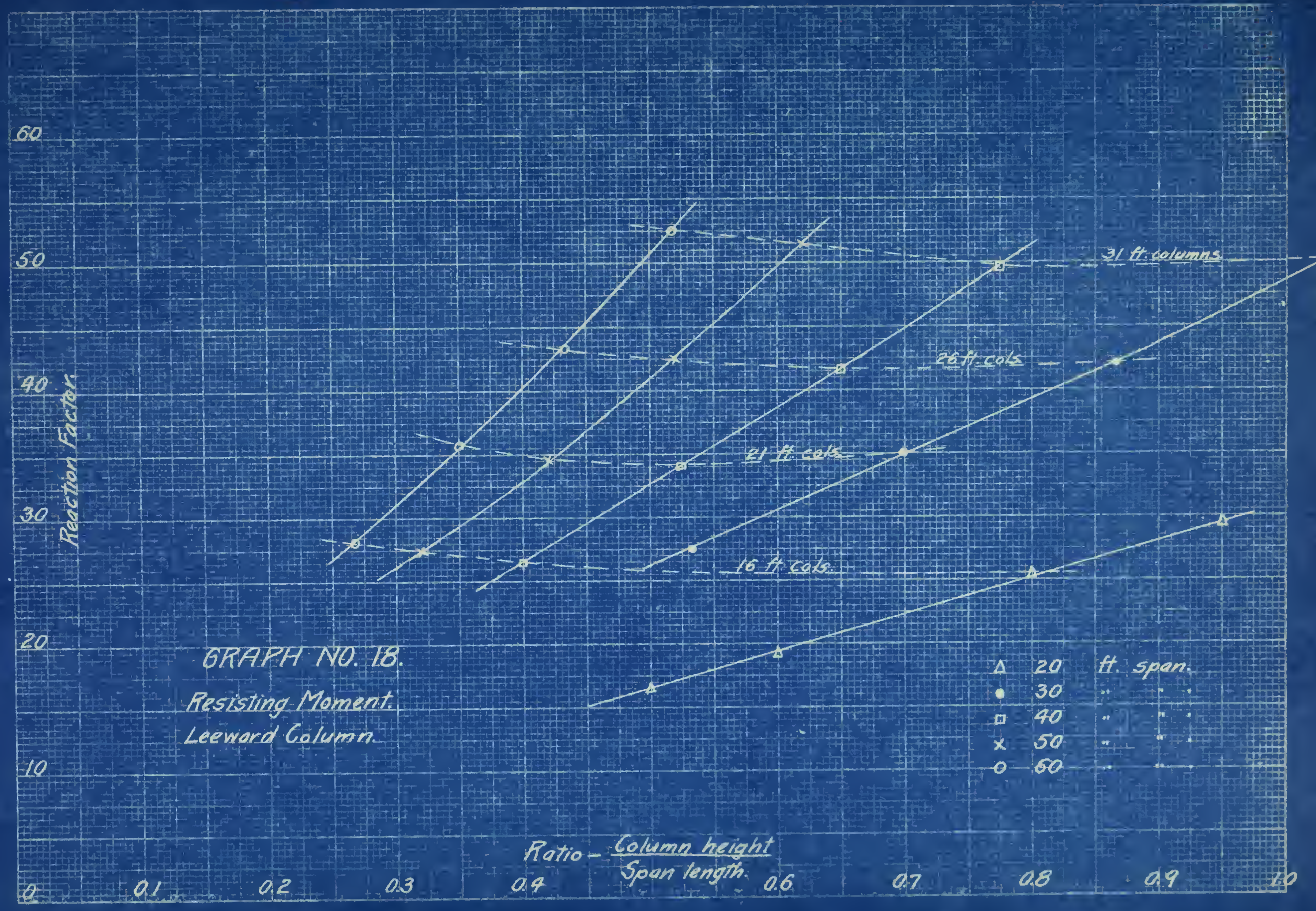

GRAPH NO. 13.
Vertical Reactions.
Windward Column.
Fixed Columns.
Hinged Columns.
Reaction Factor.
+0.5
+0.4
+0.3
+0.2
+0.1
0
-0.1
-0.2
-0.3
-0.4
-0.5
0
0.1
0.2
0.3
0.4
0.5
0.7
0.8
0.9
1.0
Ratio - Column height / Span length
△ 20 ft. span.
● 30 " "
⊡ 40 " "
X 50 " "
⊙ 60 " "

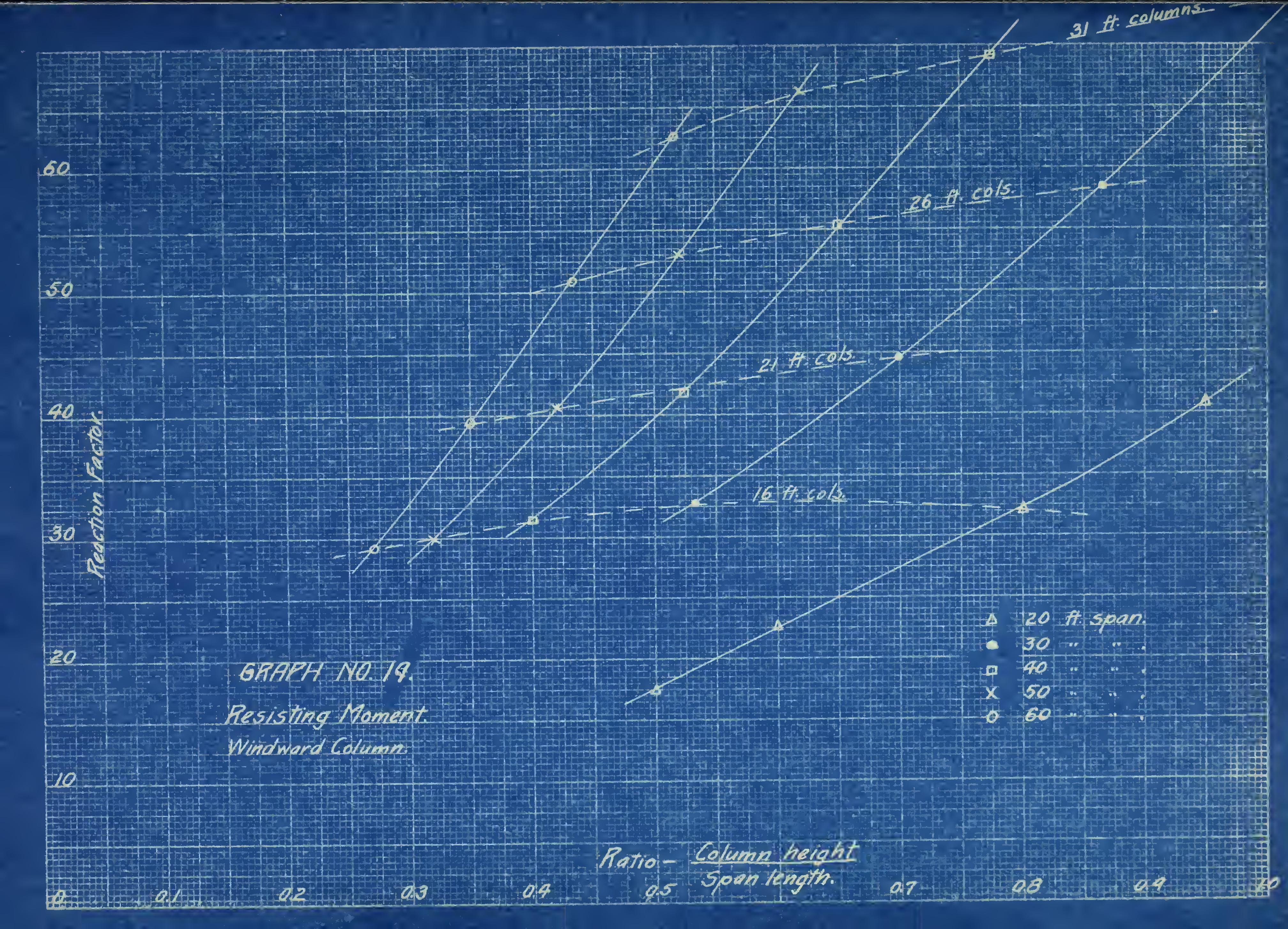

GRAPH NO. 14
Resisting Moment
Leeward Column
Δ 20 ft. span.
● 30 " "
□ 40 " "
X 50 " "
⊖ 60 " "
Reaction Factor
Column Height
60
50
40
30
20
10
0
4
8
12
16
20
24
28
32
36


GRAPH NO. 15.
Resisting Moments.
Windward Column.
Δ 20 ft. span.
● 30 " " "
□ 40 " " "
× 50 " " "
○ 60 " " "
Reaction Factor
Column Height.
0
4
8
12
16
20
24
28
32
36
10
20
30
40
50
60

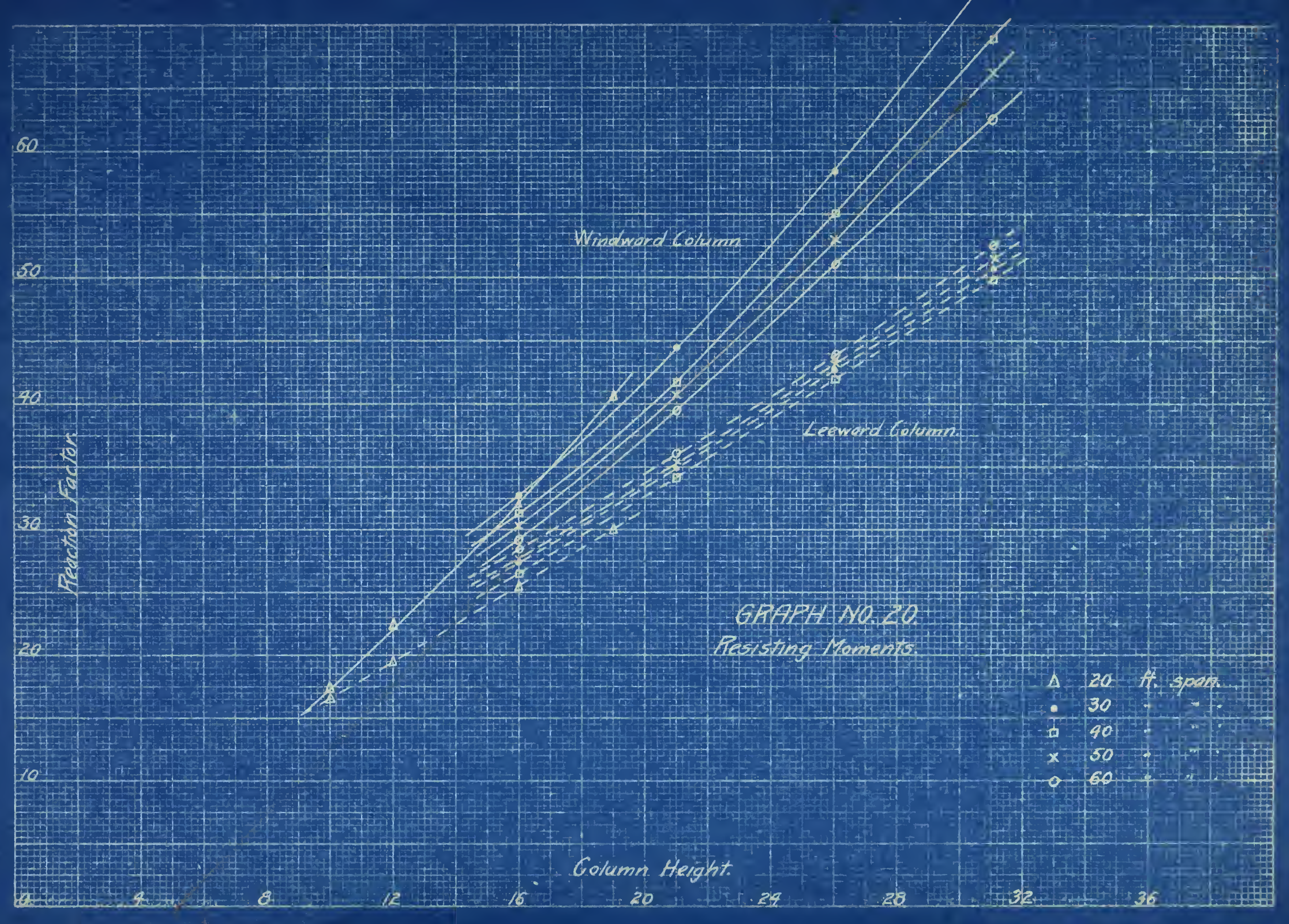

GRAPH NO. 16.
Resisting Moment.
Leeward Column.
31 ft. columns.
26 " "
21 " "
16 " "
Reaction Factor.
Span Length.
60
50
40
30
20
10
0
8
16
24
32
40
48
56
64
72


31. ft. columns.
26 " "
21 " "
16 " "
GRAPH NO. 17.
Resisting Moment.
Windward Column.
Reaction Factor
Span Length
60
50
40
30
20
10
0
8
16
24
32
40
48
56
64
72

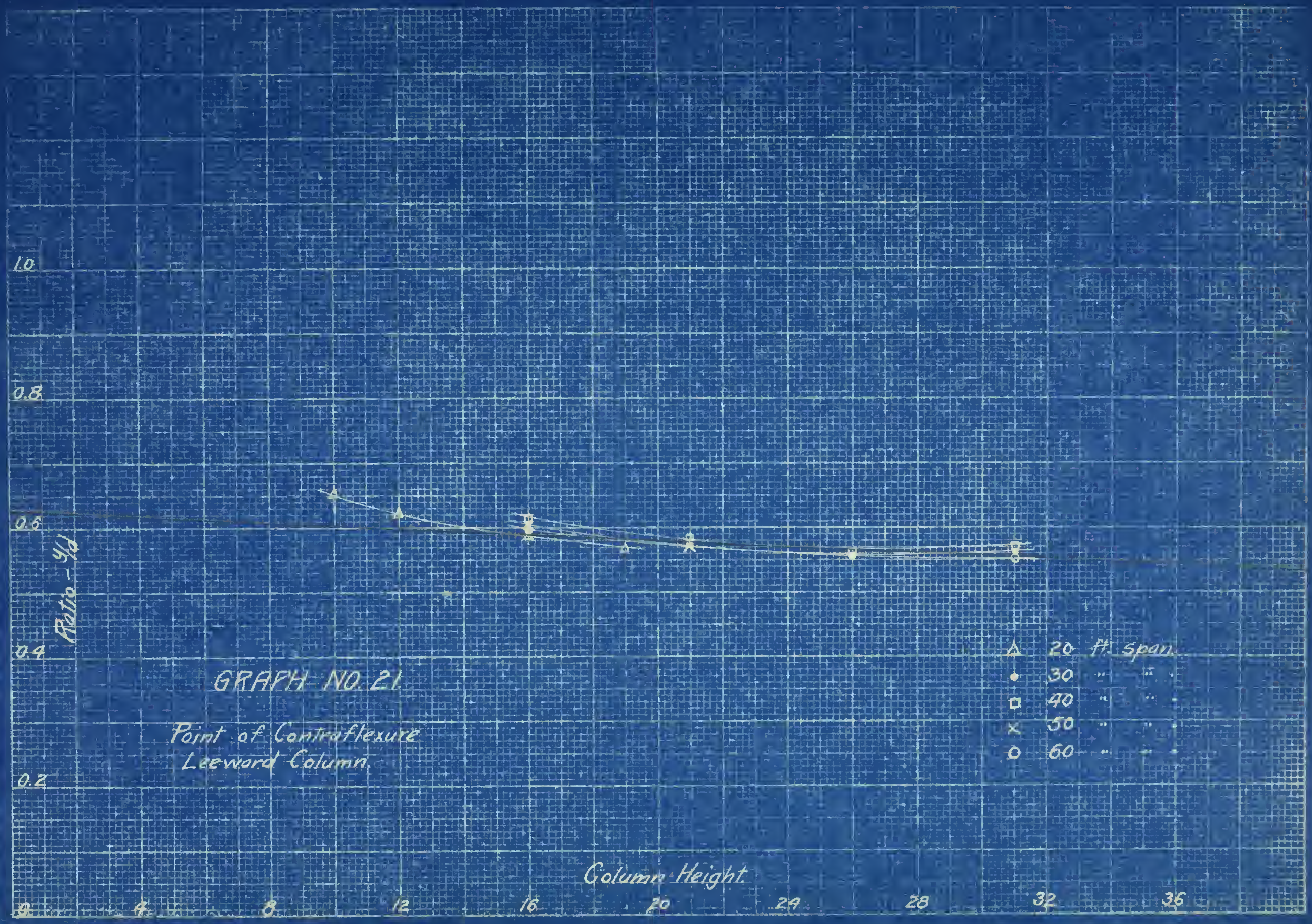

GRAPH NO. 18.
Resisting Moment.
Leeward Column.
Reaction Factor.
Ratio - Column height / Span length.
31 ft. columns.
26 ft. cols.
21 ft. cols.
16 ft. cols.
△ 20 ft. span.
● 30 " "
□ 40 " "
× 50 " "
○ 60 " "
0
10
20
30
40
50
60
0.1
0.2
0.3
0.4
0.6
0.7
0.8
0.9
1.0

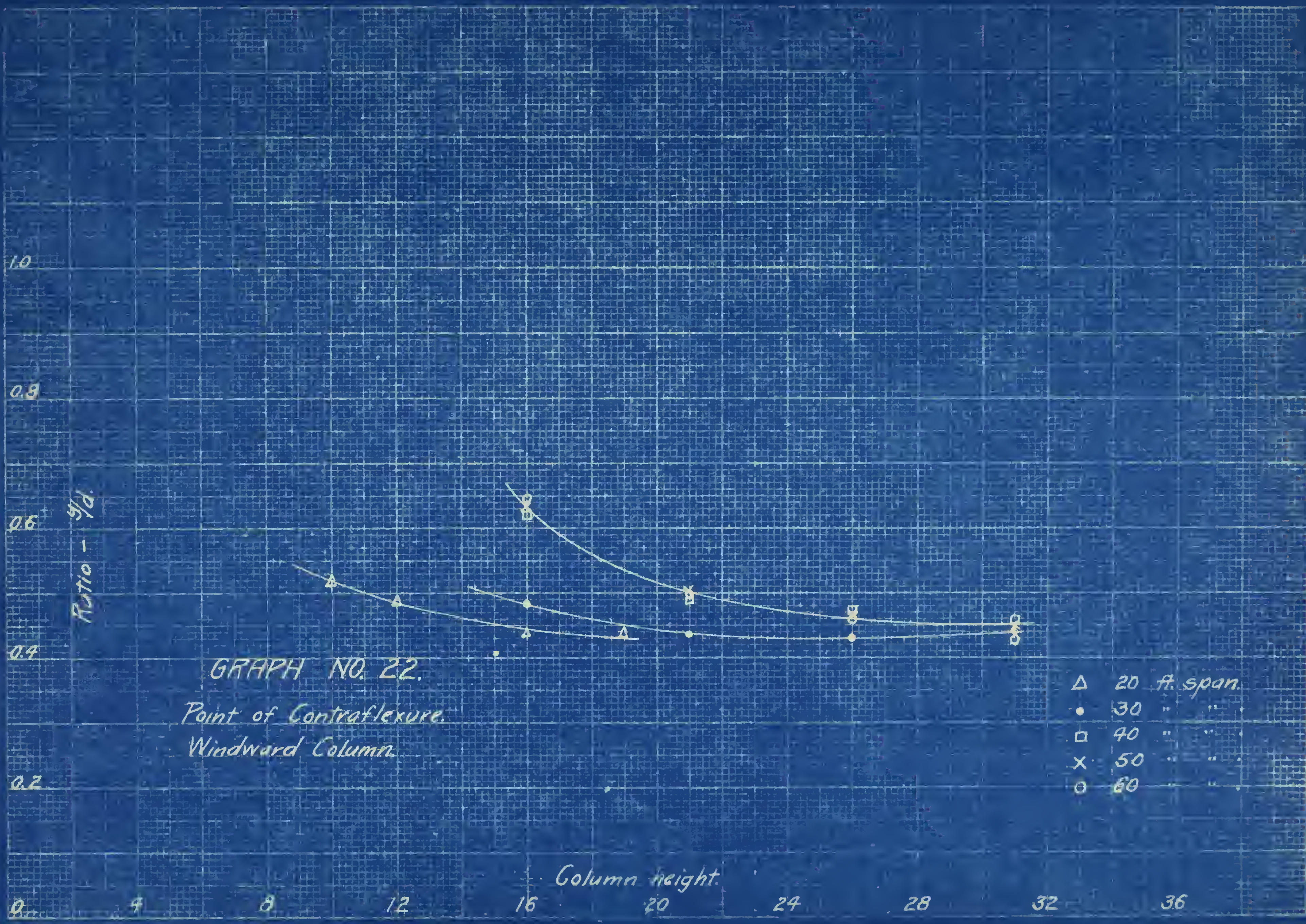


GRAPH NO. 19.
Resisting Moment.
Windward Column.
Reaction Factor.
Ratio - Column height / Span length.
31 ft. columns.
26 ft. cols.
21 ft. cols.
16 ft. cols.
△ 20 ft. span.
● 30 " "
□ 40 " "
X 50 " "
○ 60 " "
0
10
20
30
40
50
60
0.1
0.2
0.3
0.4
0.5
0.7
0.8
0.9
1.0

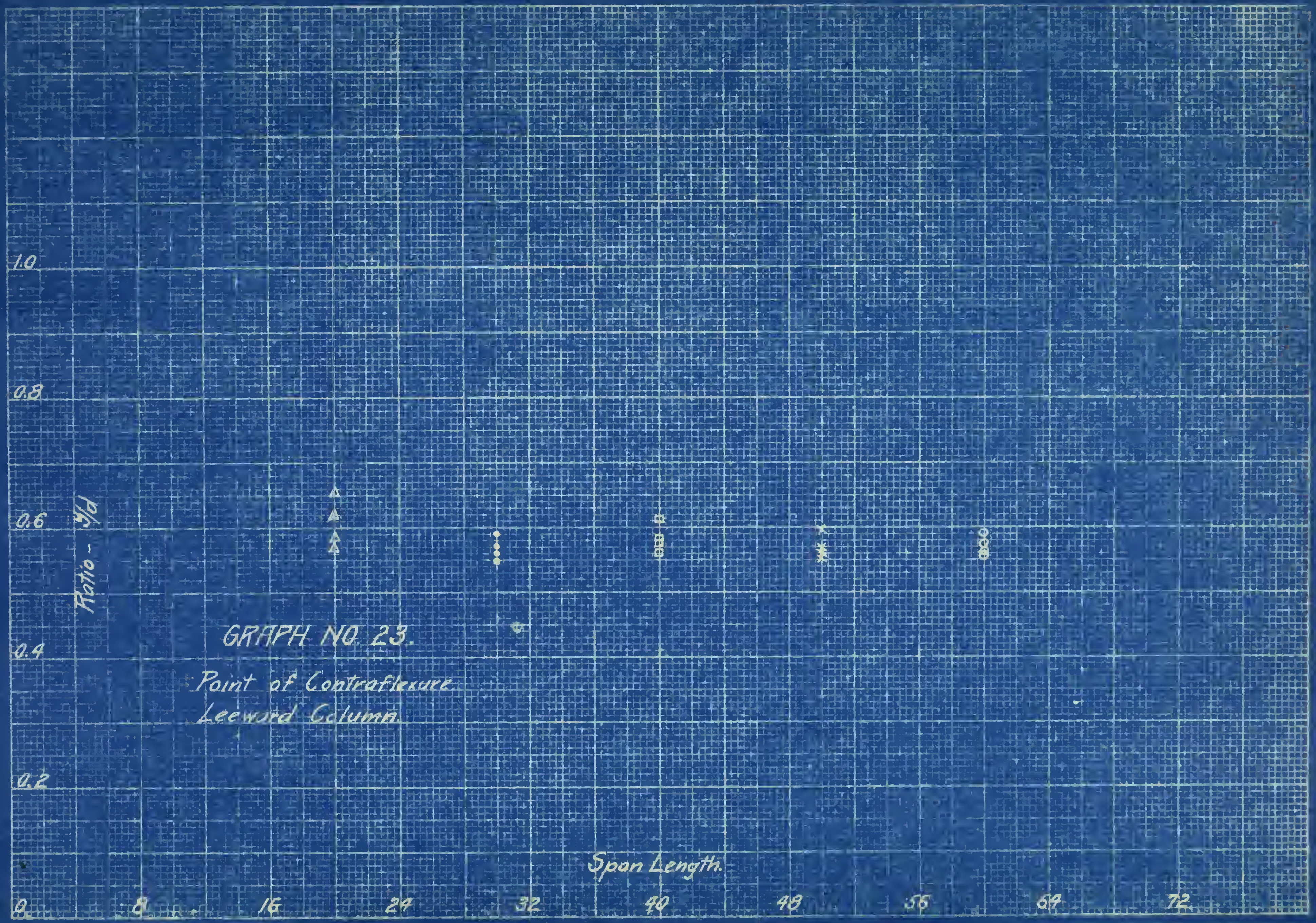


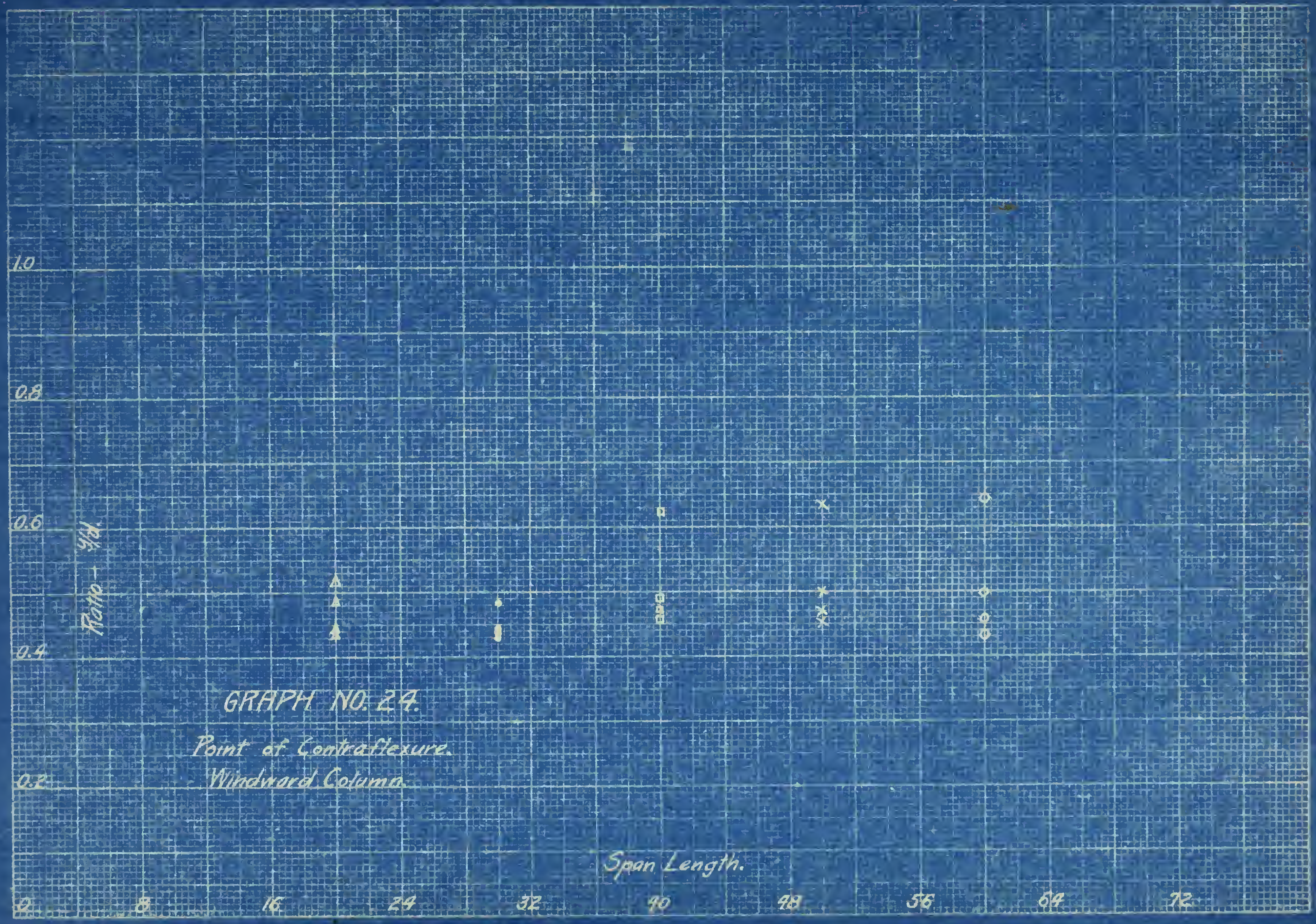
GRAPH NO. 20.
Resisting Moments.
Windward Column
Leeward Column.
Reaction Factor
Column Height.
Δ 20 ft. span.
● 30 " "
□ 40 " "
× 50 " "
○ 60 " "
10
20
30
40
50
60
0
4
8
12
16
20
24
28
32
36

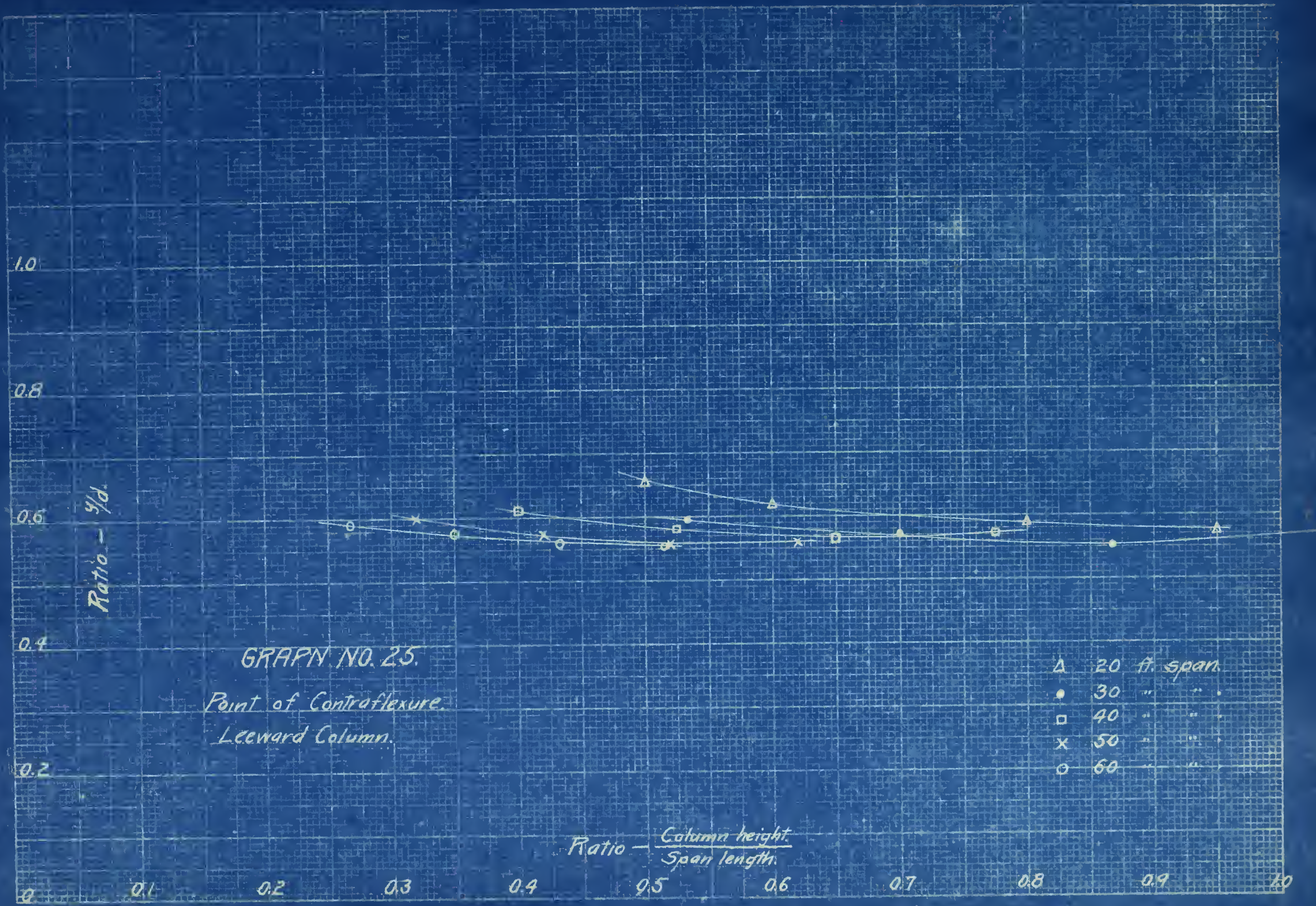

GRAPH NO. 21
Point of Contraflexure
Leeward Column.
Ratio - y/l
Column Height
1.0
0.8
0.6
0.4
0.2
0
4
8
12
16
20
24
28
32
36
20 ft. span
30 " "
40 " "
50 " "
60 " "



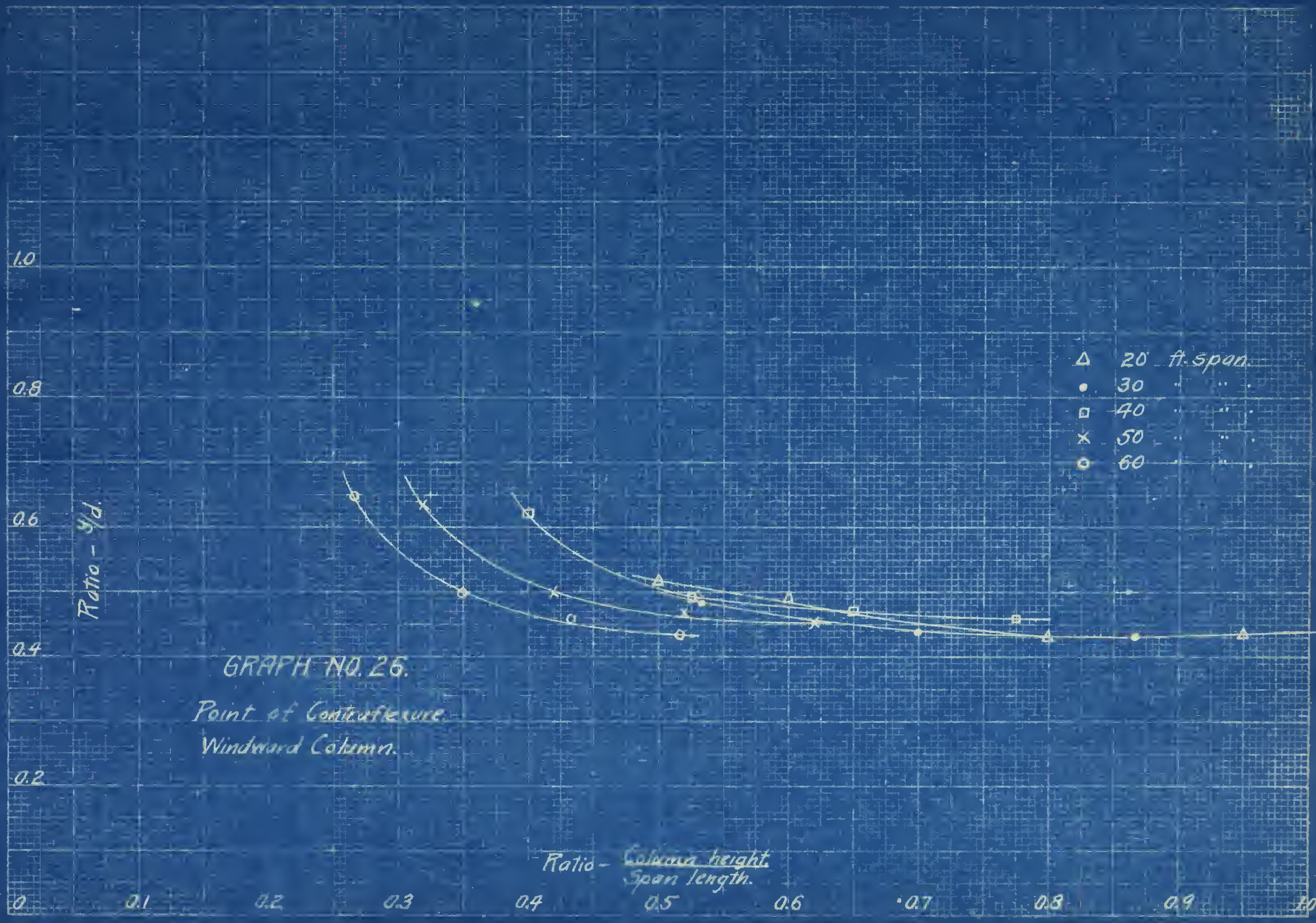
GRAPH NO. 22.
Point of Contraflexure.
Windward Column.
Ratio - y/d.
Column height.
0
4
8
12
16
20
24
28
32
36
0.2
0.4
0.6
0.8
1.0
△ 20 ft. span.
• 30 " "
□ 40 " "
× 50 " "
○ 60 " "


GRAPH NO. 23.
Point of Contraflexure
Leeward Column.
Ratio - y/d
Span Length.
1.0
0.8
0.6
0.4
0.2
0
8
16
24
32
40
48
56
64
72


GRAPH NO. 24.
Point of Contraflexure.
Windward Column.
Ratio - y/d
Span Length.
1.0
0.8
0.6
0.4
0.2
0
8
16
24
32
40
48
56
64
72



GRAPN NO. 25.
Point of Contraflexure.
Leeward Column.
Δ 20 ft. span.
• 30 " "
□ 40 " "
× 50 " "
○ 60 " "
Ratio - y/d.
Ratio - Column height / Span length.
0.2
0.4
0.6
0.8
1.0
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1.0


GRAPH NO. 26.
Point of Contraflexure.
Windward Column.
Ratio - y/d.
Ratio - Column height / Span length.
△ 20 ft. span.
• 30 " "
□ 40 " "
× 50 " "
○ 60 " "
1.0
0.8
0.6
0.4
0.2
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1.0